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CONCRETE PAVEMENT OVER POOR SUB-GRADE AT PORT NEWARK, NEW JERSEY

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Synopsis

In the spring of 1926, the immediate necessity for building 22 000 sq. yd. of permanent pavement on a comparatively fresh hydraulic fill of clay, confronted the Bureau of Streets, Newark, N. J. To meet these sub-grade conditions an extensive under-drainage plan was adopted and an 8-in., 1:1.5:3, one-course, Portland cement concrete pavement was laid in the form of slabs 10 ft. wide and 30 to 40 ft. long. These slabs were reinforced to the extent of 198 lb. of double mat, ⅜-in. bar reinforcement for each 100 sq. ft.

It is the purpose of this paper to describe the sub-grade problem, to go into the merits of the drainage and paving design, and to show that:

1.—A layer of fine granular material alone is insufficient to under-drain a pavement in the Port Newark area.

2.—Such a material on a properly shaped sub-grade, combined with a tile and broken-stone drainage system, will effect complete under-drainage.

3.—Pavements at Port Newark are subject to sub-grade defects uncontrolled by under-drainage and possible of correction only by pile supports.

4.—Next to a pile-supported pavement, the design herein described will prove most satisfactory and economical.

The Sub-Grade Problem

The principal reasons leading to the adoption of an unusual concrete pavement design in Newark, N. J., were:

1.—The character of the subsoil and underlying strata.

2.—The presence of surface water in quantity.

3.—The lack of time, limiting the operations preliminary to paving.

Note.—Written discussion on this paper will be closed in January, 1929.

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Due to the dearth of adequate and conclusive information on sub-grade preparation and the many advances made in concrete pavement, the developed design is a mixture brewed of theory, experiment, and experience.

History of Paving Site

The 22 000 sq. yd. of pavement designed to meet the conditions enumerated, were laid in the spring of 1926 at Port Newark, in response to the sudden demand for streets imposed by the rapid port development. The general layout of the site is shown in the air view, Fig. 1, and the relation of the locality to the Metropolitan Area, in Fig. 2. A brief history of the paving area, which fronts the ship channel and extends about 1 500 ft. inland, will provide a background for the description to follow.

Originally, the entire area consisted of a flat level expanse of salt meadow at a general elevation of 5 ft. above mean low water (Fig. 3). An average tidal range of 4.7 ft. kept the soft, spongy humus habitually moist. In 1917, as part of a development started several years previously, this area was raised to an elevation of about 8.5 ft., using material dredged from Newark Bay. In 1921, additional hydraulic fill was placed to an elevation of about 12 ft. Compaction and settlement of the fill and underlying strata required still more material three years later in order to maintain the elevation of 12 ft. In 1926, there was, on an average, a 9-ft. layer of clay fill the surface of which was at an elevation of 11 ft. above mean low water.

A number of wash borings in the paving site, revealed material clearly defined, but varying in thickness. Using the average depths, from which departures up to 10 ft. are found, the cross-section is about as shown in Fig. 3. The bed-rock, about 75 ft. below the surface, is a sedimentary shale of the Triassic formation common to Northern New Jersey. The 50 ft. of material (clay and sand) immediately above it, is part of the post-glacial drift. Next above this is a 10-ft. layer of mud and silt deposited by the waters entering the vast lake or bay once covering the area. Topping this layer is 5 ft. of humus, the result of the growth and decay of centuries of salt-marsh vegetation. This original soil is now covered by an average 9-ft. layer of dense red clay, hydraulically placed. Possessing all the disqualifications of clay as a sub-base, this upper layer presents the principal problem for successful road construction. A secondary, but no less important, consideration is the plasticity of the layers just beneath.

Observations and Tests on Sub-Grade

Some observations that directly or indirectly throw light on the character of the soil and sub-grade were made.

Water Content.—When the dredged material was pumped in, two features were noticeable. In the first place, most of the material was in a finely divided state held in suspension by the water, causing the fill to resemble a quicksand for months, or until the drying out was complete. In the second place, numerous rounded masses, "balled clay", came through the pipe line. On examination, the cores of these masses were found to be perfectly dry.

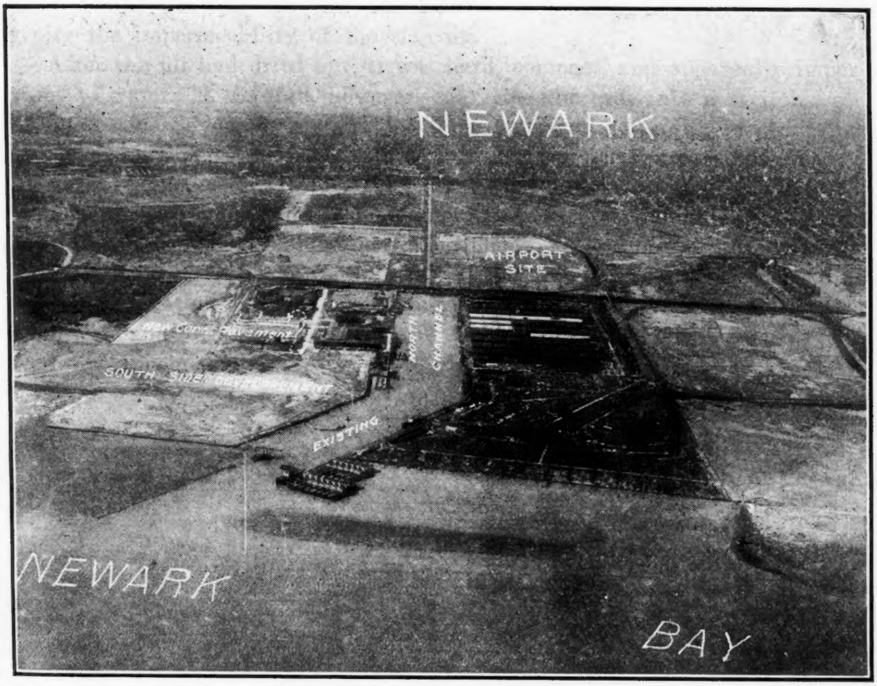

FIG. 1.—AIR VIEW OF PORT NEWARK, N. J., LOOKING WEST.

FIG. 2.—COMPOSITE PHOTOGRAPH SHOWING LAYOUT AND SITUATION OF PORT NEWARK, N. J., WITH RELATION TO NEW YORK HARBOR.

These features in themselves are not unusual in dredging experience, but they typify the impermeability of the clay fill.

After the fill had dried out, it was hard, compact, and apparently impervious to water. A rainfall, however, converted the area into a veritable sea of mud, causing water to stand for some time and softening the soil thus immersed to a depth of from 6 to 12 in. Run-off and evaporation disposed of most of the water, restoring, in time, the former hard, dry condition. These alternate conditions of high and low moisture content, resulted in a considerable change in volume as was evidenced by the cracks formed in the drying out process. These "sun cracks" varied in width and depth, depending on the penetration of the surface water and on climatic conditions. Cracks, 1 in. wide and 8 in. deep, dividing the surface into numerous small polygons, were not uncommon.

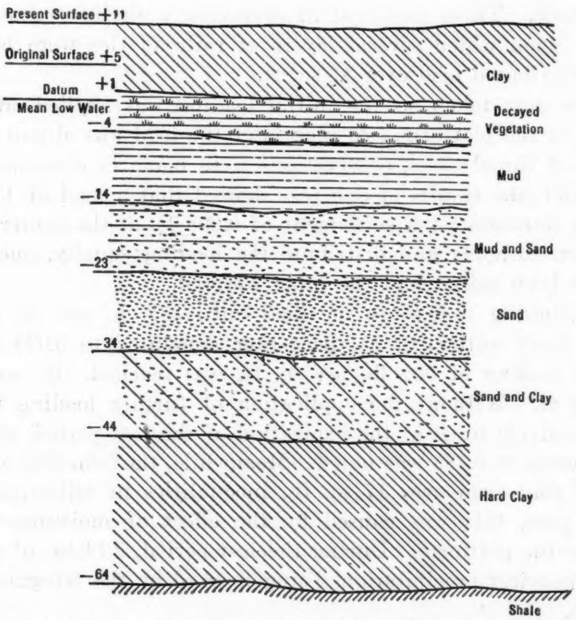

FIG. 3.—TYPICAL PROFILE OF MATERIAL OVERLYING ROCK AT PORT NEWARK, N. J.

It would appear, therefore, that little percolation, if any, takes place and that the ground-water level is not influenced by surface water. However, after a rainy period, a trench only 3 ft. deep shows seepage, whereas, in dry periods, 5 or 6 ft. may be excavated before ground-water is encountered. The water content of the soil, then, is affected to considerable depths by rain and melting snow and ice.

Behavior of Piles.—Two influences operate to make the soil unstable, the moisture which softens the surface layer and the plasticity of the under layers. For all building construction in the Port Newark area, pile foundations are required.

To attain a bearing value of 16 tons, it has been found that the average pile must be driven to about 18 ft. below mean low water. However, a wide range from this average is experienced. Numerous pile-driving tests have been made, but thus far no method has been found to predict the pile lengths required for the sites within the area. The minimum penetration may be to 12 ft. below mean low water in one place, while not more than 20 ft. distant, a maximum depth of 32 ft. will be necessary to reach solid bearing. Pile-bearing value is largely dependent on skin friction, yet from the road builders' standpoint these values suggest that the strata beneath vary to such an extent in thickness, moisture content, and physical composition, that they influence the surface stability even for light loading.

Results of Bearing Tests.—In an endeavor to determine just what reliance could be placed on the light load-carrying capacity of the soil, two bearing tests were made. These consisted in preparing a shallow excavation with a level bottom upon which several layers of railroad ties were laid, imposing an equally distributed load over 17 sq. ft.

Elevations were taken, as the initial loading was applied, at each of the four corners of the platform. A week later the load was almost doubled, and for a period of three weeks, observations were made to determine the settlement. In brief, the results of one test showed that a load of 174 lb. per sq. ft. caused an immediate net settlement of 0.008 ft. at the center; but despite the care taken to apply and distribute the loading equally, one edge of the platform had been raised and the other lowered.

After continuing to add to the load until 352 lb. per sq. ft. had been reached, the total settlement at the center amounted to 0.039 ft. When a week later a loading of 652 lb. per sq. ft. was reached, the settlement had become 0.172 ft. Without the application of further loading the platform settled progressively until at the expiration of the test period, of four weeks, a total settlement of 0.185 was noted. Meanwhile, the behavior of the corners demonstrated that even over 17 sq. ft., inequalities in soil-supporting power existed. In part, this was caused by the effect of moisture in softening the soil under the platform. During the test period, 2.14 in. of rain fell, the times of the heaviest precipitations being marked by the irregular settlement of the corners.

Soil Movements.—The movement of the top few inches of the clay, merely indicated by this test, was conclusively shown when cuts were made through a 3-ft. cinder railroad embankment placed on the clay fill. The clay surface under the center line of the track (Fig. 4) was depressed 6 to 18 in. and under the edges of the ties, 4 to 8 in., while outside the toes of the slopes it was forced upward 4 to 8 in. The extent of this movement was cumulative; that is, the deeper the clay depression became, the greater the volume of surface water collected and the more penetrating the moisture. Using the field test methods advocated by A. C. Rose,* Assoc. M. Am. Soc. C. E., the moisture equivalent of the subsoil was found to average about 50% and the linear shrinkage, 7 per cent.

* *Proceedings,* Am. Soc. C. E., January, 1928, Papers and Discussions, pp. 125 and 132.

The influence of each of the two plastic layers just beneath the fill, was distinct. The meadow strata are very elastic and act as a cushion upon which the clay fill rests. Application of a light load to a sample of this spongy material results in considerable compression and a release of pressure causes almost complete restoration of volume. Filling over this layer compresses it as much as 5 ft. in places and slight additional settlement may occur for several years. The mud and silt layer, being in a semi-fluid condition, tends to flow in the direction of least resistance. The effect of this layer is to equalize pressures imposed on it. On a large visible scale this equalization occurred during the placing of the hydraulic fill. As the material rose in back of the bulkhead, acres of bay bottom, for ¼ mile, or more, seaward (formerly 2 and 3 ft. under water), emerged. This influence on surface structures cannot be disregarded, particularly when they are adjacent to the ship channel or bay front where the mud and silt layer is less confined. Imposition or removal of heavy concentrated ground loading, or the variable water loading due to tides, may operate, through this semi-fluid layer, to set up stresses in the clay crust.

Conclusions for Sub-Grade Study

The foregoing studies and observations revealed so many indeterminate factors that a positive design, based on the usual assumptions, seemed impossible. The principal sub-grade factors that influence a pavement may be analyzed, however, and listed according to cause, effect, and remedy, as follows:

I.—Presence of Moisture in Clay Subsoil:

(*A*) Cause:

1.—Physical properties inherent in soil, such as,

(*a*) High moisture equivalent; and

(*b*) Finely divided condition.

2.—Capillarity plus low elevation above sea level.

3.—Long standing surface water, due to,

(*a*) Impermeability of soil; and,

(*b*) Irregular contour of surface.

4.—Uniform strata of clay unbroken by veins or by adjoining strata of more permeable material.

(*B*) Effect:

1.—Lessened, variable, and uncertain bearing power due to plasticity.

2.—Changes of volume due to excessive shrinkage.

3.—Frost action.

4.—Lateral movement of the top-soil on application of the loading.

(*C*) Remedy:

1.—Draining off the surface water.

2.—Use of a layer of fine granular material to:

(*a*) Decrease the height of capillary moisture;

(*b*) Quickly remove water from surface; and,

(*c*) "Cushion" the effect of frost "heaving".

3.—Distribution of the loading over large areas.

II.—Movement of Sub-Grade Soil Layers:

(*A*) Cause:

1.—Additional compression of the meadow layer due to the added loading.

2.—Further consolidation of the clay fill.

3.—Equalization of pressures due to,

(*a*) Weight of tidal prism;

(*b*) Variation in the loading and of the surface.

(*c*) Deep excavations into the mud and silt layer.

4.—Inequalities in the thicknesses of the layers.

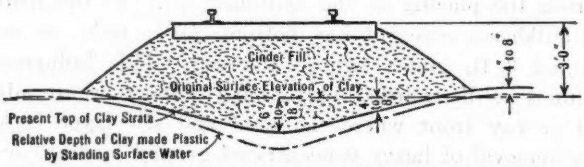

FIG. 4.—LATERAL DISPLACEMENT OF MOIST CLAY SUB-GRADE UNDER RAILROAD EMBANKMENT LOADING.

(*B*) Effect:

1.—Permanent settlement.

2.—Variable vertical movements.

3.—Upheaval of the clay crust at planes of weakness.

(*C*) Remedy:

1.—Pile foundation for the pavement.

2.—Use of substantial large sized paving units which would remain intact and keep their "riding qualities" while undergoing displacement due to sub-grade movements.

LESSONS FROM EXISTING PAVEMENT

In 1925 the construction and maintenance of roads and streets at Port Newark was transferred from the Bureau of Docks to the Bureau of Streets. At this time the only street in the south side area (Figs. 1 and 2), was a 36-ft. granite block pavement, 1 400 ft. long, laid in the spring of 1924.

Physical Features.—This roadway consisted of 4 by 5 by 10-in. blocks laid on a 1-in. sand-cement cushion with joints of asphalt and sand, all on a 6-in., reinforced, 1:2.5:5, Portland cement foundation. Naturally, before the design and construction of additional streets, considerable study was given this one.

The base was reinforced with ⅝-in., deformed, square rods, on 6-in. centers transversely and 20-in. centers longitudinally, placed 2 in. above the bottom of the concrete. The pavement was laid on a 0.33% grade and to a 4-in. crown, with one edge on the center line of the proposed 72-ft. width, the curb being about level with the surrounding ground. A depth of about 12 in. was excavated to provide for a layer of steam cinders. No particular grading or shaping of the clay under-grade was attempted, the spring rains creating a deep layer of mud upon which cinders were placed and compacted as rapidly as excavation progressed. The result was a layer of cinders, 6 to 18 in. deep, spread and compacted over an irregular clay surface. No under-drains were placed.

Stability.—Considering the subsoil drainage feature only, complete neutralization of the poor sub-grade was not accomplished. While it has been quite recently stated that "a poor sub-grade material can be transferred into a good sub-grade by the addition of sufficiently fine granular material or by the use of a layer of suitable granular material,"* this alone does not appear to effect the desired transformation at Port Newark.

It is true, the cinder course permits rapid percolation, keeps capillary water low, and acts as a cushion between the pavement and the volumetric and frost movements of the sub-grade; yet its efficiency is partly destroyed by the irregular contour of the under-base. Thus, when the clay sub-grade is recessed for the pavement, water will gravitate to this trench and remain there, giving rise to lateral soil movement as found beneath the 3-ft. railroad cinder embankment previously mentioned. Irregularities of the clay, forming water-pockets, will aggravate the condition. Where a sufficient area is thus weakened beyond the ability of the concrete base to sustain the surface loading, failure might be expected. Examination of the pavement indicates that this may have occurred. The surface has many crosswise waves, the close spacing of the transverse reinforcing rods having doubtlessly prevented even more serious dish-shaped depressions. A more happy choice of reinforcing bar design might have still further aided in resisting the sub-grade forces.

As bearing out the assertion that sub-grade movements affect a pavement (equalization of pressure and other causes being ascribed), a comparison of elevations taken immediately after paving with similar readings taken in 1925, shows that, in general, a settlement (as much as 12 in. in some places) has occurred. A short section revealing a slight rise along one side and the irregular settlement over sections (as distinguished from localized areas), indicates the variable effect of sub-grade movements.

The relation between the location of such pavement movements and the many warehouses and other surface loadings has not been investigated. The fact that all buildings were erected subsequent to paving is, however, a starting point for such an investigation.

Under-Drains and Their Effect

In the spring of 1926, when paving became necessary, the same conditions as existed for the 1924 paving were present, namely, a saturated top surface of clay, and standing surface water due to frequent rains. Two types of under-drains were used—a single arterial line in the center with laterals, for pavements of less than 50 ft. in width (Fig. 5); and a double arterial line, one under each gutter, with laterals, for pavements of greater width (Fig. 6). The arterial lines are 6-in. tile-drains (none extends more than 400 ft. before entering a surface-water catch-basin or open ditch) laid to a 0.50% minimum grade, recessed in the clay under-grade, and completely surrounded by broken stone. Tees are placed every 50 ft. from which broken-stone drains through the clay run out to the edge of the pavement.

* "Recent Conclusions in Highway Research," by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., *Roads and Streets*, March 1925, p. 458.

Tarred paper around joints, between the cinders and the broken stone, and other features common to such construction were included. In addition to the tile and stone drains, the clay under-grade was carefully shaped to direct water toward the arterial drains.

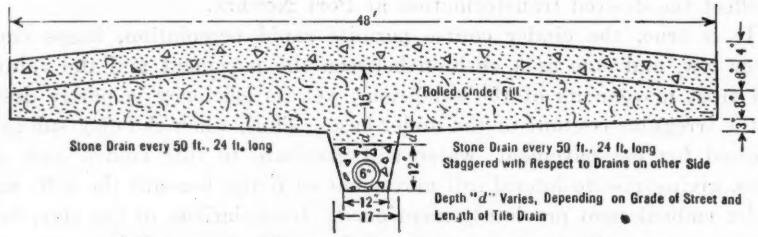

FIG. 5.—CROSS-SECTION OF 48-FOOT CONCRETE PAVEMENT.

The value of these under-drains first showed itself when a dry clay under-grade was obtained upon which a layer of cinders could be carefully laid and compacted in place. After a pavement is laid their value is, of course, not so apparent. In view of the paradoxical results as to moisture content obtained under drained and under-drained sections of the Bates Test Road,* it might appear that tile drains fail to live up to expectations. The common assumption is that a tile pipe will drain a width of about six times its depth in clay soils, and fifteen to twenty times its depth in porous soils. When this assumption is made the fact that "time" is an important element is generally overlooked—particularly where clay soils are concerned; for, if drainage becomes operative only on saturation of the clay, then its value is nil. On the other hand, if a rate of flow through the soil is established shortly after the surface is wetted, then the system has merit.

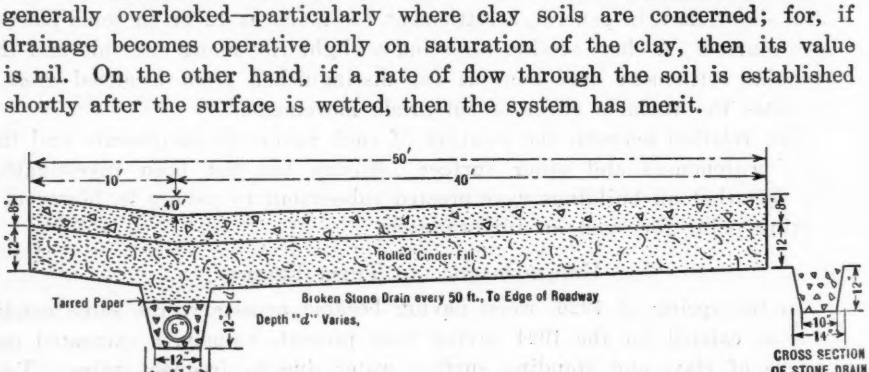

FIG. 6.—CROSS-SECTION OF ONE-HALF OF 100-FOOT CONCRETE PAVEMENT.

Field studies would best determine the behavior of water on passage through a clay subsoil. However, as an index to what may occur, the experience of the writer in making the moisture equivalent tests will serve. Samples of the subsoil were gathered. While obviously the clay contained moisture, the masses were hard to break up and no dampness was visible; not even did the paper on which the samples were laid, become moistened.

* "Tentative Conclusions on Design of Concrete Surfaces", by Clifford Older, M. Am. Soc. C. E., *Roads and Streets,* May, 1924.

So little water was added to produce a "shiny" surface on the test samples of the clay that the writer decided to dry out the unused portions of several samples and thus, by weight, determine what moisture was originally present. The results showed that each sample in its original state contained about 20% of water. If a mass of 25 lb. of such clay can be removed from the ground and exposed without drying out, it is evident that the most closely spaced tile drains will likewise fail to remove the moisture. While from the appearance of the clay this condition did not affect its bearing power, yet, evidently, much more moisture would accumulate before "seepage" or drainage would begin.

Assuming, then, that the tile and stone drains are ineffective in themselves in draining water through clay, they still are important in removing the water brought to them by the sloping clay under-surface. To recapitulate:

1.—The porous cinder layer rapidly removes water from beneath the pavement;

2.—This is aided by the herring-bone under-drainage system; and

3.—Water otherwise saturating the clay to disintegration is drained, as surface water, into the "broken-stone gutters."

Concrete Pavement Provided

Design.—Consideration of various types of pavement finally led to the selection of concrete as best suited to the sub-grade conditions at Port Newark. The developed design consists of a 1:1.5:3 Portland cement, reinforced, concrete pavement laid to a uniform thickness of 8 in. The double mat reinforcement is composed of $\frac{3}{8}$-in. deformed round bars assembled so that the lower mat is supported 2 in. above the bottom of the pavement and the upper mat is separated therefrom by $3\frac{1}{2}$ in. The lower members are spaced 8 in., both transversely and longitudinally; the upper transverse bars are on 8-in. centers, while the longitudinal bars are on 16-in. centers. The reinforcement extends to within 2 to 6 in. from the edges of the slabs, adjacent mats being properly lapped and the other usual practices observed.

All joints are vertical, with $\frac{3}{4}$-in. dowel-pins connecting the slabs across transverse joints. At all slab corners and angles, $\frac{3}{8}$-in. bars, 6 ft. long, were bent outward in the form of a U (sides at an angle of 22°) and placed in the top mat to combat corner failures. The specifications under which the work was performed, embody many of the proposals advanced by the committees on concrete pavement specifications of various technical societies. Daily laboratory tests checked the quality of the materials used and the strength of the resulting product. In the field, the slump test was used to control the consistency of the mix.

In scrutinizing any concrete pavement design, examination in the light of recent investigation is desirable. Each feature of this design will be so considered.

Mix.—The 1:1.5:3 mix, with trap rock as the coarse aggregate, appears to represent the greatest strength and the least variability for road-building

purposes. Accelerated wear tests conducted by the U. S. Bureau of Public Roads reveal that a 1:1.5 mortar with trap-rock aggregate provides one of the most resistant surfaces to abrasion.* It is particularly fitting that this mix be used at Port Newark where heavy trucking is the principal class of traffic.

Thickness.—Students of recent development in design will be inclined, no doubt, to view a uniform thickness as a less advanced feature than the remainder of the design. It is usually assumed that a wheel load 3 ft. from the pavement's edge will do no damage. This being the condition at Port Newark, there is no reason for increasing the thickness of these pavements. Traffic is inclined to avoid the edge (on the 100-ft. streets) due to the depressed gutter lines, and likewise on the 48-ft. streets, due to the adjacent poles or other like features.

Reinforcement.—Recent studies† made public by the Highway Research Board of National Research Council, show that by increasing the reinforcement, a reduction in cracks is obtained more economically than by increasing the concrete thickness. Specifically, 170 lb. of bar reinforcement per 100 sq. ft. (50% each way) causes a reduction in combined transverse and longitudinal cracks equal to an added 2-in. center thickness of concrete. In the design used at Port Newark there is 169 lb. of reinforcement symmetrically placed, the added alternate transverse bars in the upper mat making the total reinforcement 198 lb. per 100 sq. ft. It would follow, then, that the effectiveness of at least 2 in. of added concrete is obtained with respect to crack reduction.

Furthermore, the report states, for equal weights of reinforcement small bars closely spaced are more effective in minimizing crack development than larger bars. At Port Newark with $\frac{3}{8}$-in. bars, 8 in. apart, it is hoped that a proper distribution has been achieved.

A pavement slab design cannot be analyzed with as great a degree of certainty as the usual floor-slab, yet there are reasons why the same kind of an analysis may be applied. It may be desirable, for example, in a comparative study, or in determining the capacity of the slab to bridge newly back-filled utility trenches; or, as in the present case, in indicating the extent to which sub-grade failures will be harmless and in suggesting the possible piling required where total sub-grade failure is expected.

Computation of Strength

Where only occasional sub-grade failures are expected, pavement slabs are not designed to carry their loads entirely unsupported since such a course would be both extravagant and unjustified. At Port Newark, localized areas of non-support might possibly occur due to partial disruption of the drainage system or minor settlements attendant on the movement of the underlying strata. When such areas do occur, the worst possible condition will not be

* "Researches on the Structural Design of Highways, by the United States Bureau of Public Roads," by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E. Vol. 88 (1925), p. 264.

† Report, Highway Research Board, National Research Council, Vol. II, 1926, p. 12.

a 36-ft. slab supported at its ends only, since such a condition would presuppose exceptionally high bearing value for the sub-grade immediately adjacent to the "failure" and the ability of the dowel-pins to withstand the resulting shear. Instead, some indeterminate lesser area of non-support will generally develop, and then the slab, as a unit, will settle to a new bearing. Impact, inertia of the slab, and the many other factors, however, may operate to cause failure of the slab before it can adapt itself to the new plane of support.

As pavement slabs are used for years following their initial failure and as so many variable and uncertain elements preclude an accurate mathematical analysis, it is not so much the "safe" carrying capacity, but the ultimate carrying capacity, that is desired. For this purpose it is proposed to use 1 500 lb. per sq. in. as the "yield point"* of the 1:1.5:3 concrete; 40 000 lb. per sq. in. as the yield point of the steel; and $n = 12$.

Taking a longitudinal section of the slab, 8 in. wide, and analyzing it as a continuous beam by straight-line formulas,† and neglecting for the present the upper mat (Fig. 7(a)):

$$
\begin{aligned}
a_s &= 0.1105 \text{ sq. in.} \\
p &= 0.0023 \\
k &= 0.2089 \\
j &= 0.9304 \\
kd &= 1.25 \text{ in.} \\
jd &= 5.58 \text{ in.} \\
M_s &= 24\,643 \text{ in-lb.} \\
f_c &= 880 \text{ lb. per sq. in.}
\end{aligned}
$$

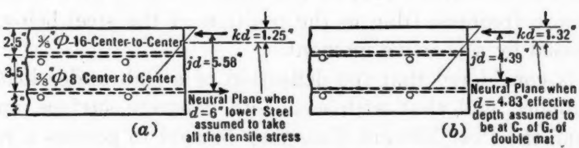

FIG. 7.—ANALYSIS OF REINFORCED CONCRETE LONGITUDINAL PAVEMENT SECTION.

An examination of Fig. 7 shows that the upper steel falls below the neutral plane.

Treating the entire steel system as taking the tensile stress in the beam at a depth equal to the distance of its effective center of gravity below the surface (Fig. 7(b)), then

$$
\begin{aligned}
d &= 4.83 \\
a_s &= 0.1658 \\
p &= 0.00429 \\
pn &= 0.05148 \\
k &= 0.273 \\
j &= 0.909
\end{aligned}
$$

* See *Bulletin No. 5*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

† Symbols and analysis follow the Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

$$kd = 1.32 \text{ in. (or 1.18 in. above the top steel).}$$

$$jd = 4.39 \text{ in.}$$

$$M_s = 27\,370 \text{ in-lb.} = \frac{27\,370}{12} \text{ ft-lb.}$$

$$f_c = 1\,260 \text{ lb. per sq. in.}$$

Using $M = \frac{w\,l^2}{12}$ as for a continuous beam,

$$\frac{w\,l^2}{12} = \frac{27\,370}{12}$$

or,

$$w\,l^2 = 27\,370$$

Then, if, $w = 500$ lb. per sq. ft. $= 333$ lb. per running ft.,

$$l^2 = 82$$

or,

$$l = 9 \text{ ft.}$$

Thus, a load of 400 lb. per sq. ft. in addition to the weight of the slab may cause failure due to bending stresses, when the unsupported area has a width equal to that of the slab and a length of 9 ft. The analysis of a transverse section in a like manner will reveal a greater resistance to failure because of the additional steel in the top mat. A cursory examination shows that a concentrated wheel load of 7 tons will probably cause failure of the slab over an unsupported area the width of the slab and about 4 or 5 ft. long.

Where negative bending moments occur, the top steel should take the tensile stresses in the upper part of the slab. Failure under such a load will be perhaps more frequent (due to the position of the steel below the surface) than that caused by a positive moment.

When it is considered that the deflection of a slab under loading may add sub-grade support, and that with a smooth concrete surface, impact is kept at a minimum, the Port Newark slabs may be said to possess a relatively high resistance to failure—but should failure occur, a separation into rather large square sections will result, with the steel stressed beyond its yield point, but still linking the sections together.

If a pile foundation is adopted to replace efforts at under-drainage, it will be necessary to re-design the slabs, decreasing the size and increasing the reinforcement in order to transfer the surface loading to the piles. Use of 6 by 7-ft. slabs, 8 in. deep, with about 270 lb. of steel per 100 sq. ft. and rows of 30-ft. piles, 8 to a "bent", with the bents 6 ft. apart, might be the principal elements of a possible design adopted. Whether the use of the Compressol System would be as suitable and more economical has not been investigated as yet by the Newark Bureau of Streets.

Comparison of Paving Costs

Itemizing the approximate actual cost per square yard of both types of pavement and under-drainage laid at Port Newark, and estimating the probable cost of a pile-supported pavement, the results are as follows:

Granite Block (Actual Cost):	
Pavement on 6-in. concrete base (including 6 in. of excavation)	$7.38
Reinforcement (about 31 lb.)	1.40
Excavation (average)	0.70
Cinder fill (average)	0.90
Average cost per square yard	$10.38
Reinforced Concrete (Most Recent of the Three Contracts Let):	
Pavement complete	$3.57
Excavation (average)	0.84
Cinder fill (average)	1.60
Tile-drain (average)	0.20
Stone-drain (average)	0.40
Average cost per square yard	$6.61
Pile-Supported Pavement (Estimated Cost):	
Pavement slab	$4.50
Pile foundation	7.00
Cinder fill	1.60
Excavation	0.80
Average cost per square yard	$13.90

Leaving aside the cost of surface drainage, gutter curbing, etc., the reinforced concrete pavement used at Port Newark appears to be the more economical in comparison with the granite block, and the more likely to prove satisfactory. Should it prove to deteriorate rapidly, the more than twice as expensive pile-supported pavement may receive serious consideration as a substitute.

Results to Date

As the pavement is of comparatively recent construction, no time-proven conclusions as to its effectiveness can be drawn. However, interesting observations have been made.

Sub-grade movement has occurred and displaced the pavement, noticeably, at three locations. In all cases it is at slab junctions and has in no wise affected the integrity of the slabs. At the maximum depression, water 4 in. deep is pocketed. Since numerous track crossings prevented the adoption of steeper grades, relatively slight slab settlement causes these water pockets to form.

Examination of the tile-drain outlets during rains shows them to be carrying a considerable volume of water during and after such periods. What settlement has occurred is believed to be due to the major movements of the under-grade layers and not to movements arising from moisture-laden subsoil. Check elevations recently taken over the pavement substantiates this belief in revealing, over considerable areas, elevations differing from those to which the concrete was originally laid. The ability of the slabs to withstand displacement of this character or adopt themselves to it, while under traffic, seems, thus far, to justify the design used.

Conclusion

At present, considerable study is being given to subsoil problems. As heretofore, the actual service experiences will apply the final test to the various solutions advanced and will point the way to still further improvements. Where a more or less experimental or novel design is used, it is believed an account of the surrounding circumstances will be of interest to engineers confronted with similar sub-grade conditions. No claim is made that the Port Newark pavement is the best design possible nor that it is entirely original, since the experience of others in the highway construction field has been freely drawn upon; but it is one in which the City of Newark reposed sufficient confidence to expend $148 000. If the foregoing account has in some degree portrayed the foundation of such confidence and contributed thereby to the advance of paving science, its purpose has been accomplished.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

COUNTY SEWER DISTRICT WORK IN OHIO AND ASSESSMENT OF COST ACCORDING TO BENEFITS*

By E. G. Bradbury,† M. Am. Soc. C. E.

Synopsis

Under Ohio laws, county sewer districts may be created and sewers and water mains may be constructed therein, the cost being defrayed by assessments levied on property benefited. More than 2 000 miles of construction have been executed under these provisions, largely in suburban or rural areas.

The character of the territory served involves special problems of design both as to boundaries of districts and provision for future development. The writer's basis of estimating proper sewer and water main capacities is described.

In the plan for the apportionment of assessments for trunk sewers in districts only partly sewered, the fundamental basis is the enhancement of value of the property affected, by reason of the improvement. The actual effect on market price cannot be predicted with accuracy, but a theoretical enhancement is worked out. The factors used are:

1.—The future cost to any part of the district for additional trunk or main sewers, for which a present credit is allowed.

2.—The estimated period in the future when service will be given to each part of the district, and an adjustment on present worth basis.

3.—The relative value of the land, a factor being used proportionate to the square roots of appraised values.

Similar methods are used in case of storm sewers and water mains, subject to special considerations due to the character of each.

Note.—Written discussion on this paper will be closed in January, 1929.

* Presented at the meeting of the Sanitary Engineering Division, Columbus, Ohio, October 19, 1927.

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History of Legislation

For a number of years the County Sewer District Law of Ohio has been a unique piece of legislation. Until recently, if the writer is correctly informed, it has represented the only attempt to provide for sewer and water facilities in unincorporated suburban and rural communities by laws of general application. One or two other States, however, now have laws of somewhat similar character.

This law is not to be confused with the "Sanitary District Act of Ohio," which authorizes the creation by the Court of Common Pleas of Sanitary Districts, including two or more political subdivisions, and the construction therein of main sewerage and water supply works for the service of the several units of such District, as the latter Act is designed for the service of contiguous or near-by municipalities, and makes no provision for local service.

The Ohio County Sewer District Law* dates back to 1911, when a crude and insufficient enactment sought to provide for the creation of county sewer districts and the construction of sewers therein. This law was soon found to be faulty and was amended in 1913 to a workable form. Some years later, further sections were added, authorizing the construction of water supplies in sewer districts and additional amendments have been passed from time to time.

An apparently radical defect in these laws, in that they did not provide for proper notice and hearings prior to the construction of improvements, was developed by litigation in Logan and Trumbull Counties within the past two years, the Courts in both cases indicating doubt as to the propriety of this procedure although failing actually to deny the constitutionality of the sections criticized. Amendments passed by the last session of the General Assembly provide in detail for such notice and hearings and the statute is now believed to be sound.

The Existing Law

As the law now stands, power is given to the county commissioners of any county to create, by resolution, sewer districts outside of municipalities, and to construct therein sewers, sewage treatment works, water supplies, and water mains, assessing the whole, or a part, of the cost on the property benefited. By authority of the council of any city or village, a part, or the whole, of such municipality may be included as a part, or the whole, of a county sewer district. Provision is made for joint construction and maintenance by two or more political subdivisions. The construction of utilities of this class by private interests, without the approval of the county authorities, is forbidden. The appointment of an engineer and assistants is provided for.

The details of procedure are described at considerable length. A general plan of sewerage or water supply for the entire area is the first requirement after the creation of a district. Then follow detailed plans, specifications,

* Comprised in Sections 6602–6611 to 6602–6633 of the Revised Statutes of Ohio.

estimates, and tentative assessments for such parts as it may be necessary to construct, a resolution of necessity, notices, public hearing, ratification or amendment, and the resolution to proceed. Provision is made for the issuance of bonds or certificates of indebtedness and the levying of revised assessments based on actual cost, but substantially in the same proportion as the tentative assessment.

Resulting Sanitary Development

Twenty-six of the eighty-eight counties of Ohio have acted under this law, in its various forms, and the extent and value of the work have far exceeded expectations. In an effort to secure full information, a letter of inquiry was mailed to the Sanitary Engineer of each of these counties. In the twelve counties from which replies were received there have been created seventy-nine sewer districts, with a total area of 336 600 acres. In these districts, sewers constructed or under construction total 807 miles, and water mains, 797 miles. Fifteen sewage disposal plants and six water supplies are reported in the same counties. Data from the remaining counties are not available, but it is very conservative to assume that more than 2 000 miles of sewers and water mains have been built under this procedure within the State.

Most of the work is the extension of the sewer and water systems of the larger municipalities, although quite a number of summer resorts and small rural communities have been provided with independent systems. The possibility of competent and systematic control of these matters in suburban areas is of great advantage to the cities, as inadequate design and improper construction are avoided, development is encouraged, and utilities are constructed at the entire expense of the property benefited. The city, of course, must be compensated for services in transporting and disposing of sewage and for water furnished.

In some of the counties, a city annexing territory that has been improved under this law simply takes over the sewers and water mains therein without compensation, and continues their operation. In others, a plan has been developed by which the city pays present value of these improvements when annexations are made. This is a question to be studied in each case, as the proper solution depends on how the city finances its own work of the same character. It is often a complex and many-sided problem.

The collection of interest on deferred installments of assessments at a rate equal to that on bonds sold to meet cost of construction, and a provision of the law that maintenance assessments may be levied as necessary, make possible the operation of sewer and water systems for rural communities much smaller than can be provided for by any other method. One water supply in Franklin County, consisting of a deep well, pumps, an elevated tank, and a distribution system, was operated for several months with twelve service taps and has never had more than seventy-five customers, yet water is sold at 35 cents per 1 000 gal., plus a service charge of 50 cents per month, and after four years of operation the original maintenance fund is intact and a small surplus has been accumulated. This is possible because there is no interest charge included in the water rates and a large replacement fund need not be

built up, as future contingencies can be met when they arise by further assessment. This project was installed at a cost to the property owners of about $100 per lot.

Extent of Districts and Sewage Flow

The determination of the proper boundaries of a sewer district presents a rather difficult problem. Correct sewer design requires that provision be made for many years in the future, and, within the time designed for, the natural growth of the community will cover much land which is now used only for agricultural purposes, although usually valued at speculative prices. Often such territory, if not included, can not be later sewered except at great expense. Many owners resent even a small present assessment, and demand that their property be omitted. When such objection becomes general over an outlying part of a district, and cannot be satisfied by explanation, it is impossible to proceed against it, and a more short-sighted policy must be adopted.

In many of the districts there is much entirely undeveloped territory. This condition renders design somewhat difficult, because of the problematical character of future development. It is reasonable to expect such areas to become very largely residential in character, although more or less industrial activity will probably follow railroad lines; ordinarily, commercial development will be limited to small local trading centers. In Franklin County the practice is to design for residential purposes only, using figures sufficiently liberal to take care of any reasonably probable manufactural or commercial demand.

The basis of sewer design used by the writer differs somewhat from the usual standards. It recognizes the probability that there may be a congestion of population in any given part of the district and the possibility that this may occur anywhere in the district, but is not likely to extend over large areas. It provides for a wider variation between average and maximum flows in small areas than in large; and it anticipates a greater ground-water infiltration than is assumed by many engineers. Although using the most careful methods of construction, the writer has never been able to prevent material increases of flow during wet periods, due to infiltration through joints and manholes, admission of surface water through manhole tops, leakage of house connections, and doubtless some water from roofs or surface water inlets surreptitiously connected. It is of some interest to note that, with manholes 300 ft. apart, an average of one leak in each manhole equivalent to a $\frac{1}{4}$-in. orifice under 2-ft. head, will amount to about 25 000 gal. daily per mile, or 750 gal. daily per acre. An average of 2 drops per sec. from each joint of a pipe sewer will total about the same figures. That is, if the leakage is to be kept down to 750 gal. per acre, the infiltration for each 300 ft. of sewer must not exceed one $\frac{1}{4}$-in. leak, or a somewhat rapid dripping from each joint.

On the other hand, the average domestic consumption in Columbus is about 50 gal. per capita daily, or approximately 50% of the average per capita use for all purposes. This figure is accurately verified by the use of water in residential suburbs controlled by master meters.

Basis of Design

In view of these considerations the following basis of sewer design has been established: (1) The future population is estimated according to a curve ranging from 40 people per acre for areas of 100 acres or less to 20 people per acre for areas of 500 acres or more; (2) it is assumed that the domestic sewage will average 75 gal. per capita daily; (3) that the maximum domestic flow will vary along a curve from 300% of the average in areas of 100 acres or less to 225% in areas of 1 000 acres or more; and (4) for ground-water infiltration, a flat rate of 3 000 gal. per acre daily is assumed. The net result of these figures is as follows:

Acres	Cubic feet per second per acre.
50 or less	0.020
51 to 100	0.019
101 to 125	0.018
126 to 150	0.017
151 to 175	0.016
176 to 200	0.015
201 to 250	0.014
251 to 300	0.013
301 to 400	0.012
401 to 500	0.011
501 or more	0.010

In applying this tabulation quantities corresponding to the total area contributing are used and not a cumulative total. It will be noted that larger capacities are provided for small areas than are advocated by many engineers. This seems desirable in view of the fact that overtaxed sewers are much more frequently found near the upper end of a system than in the intercepting or outfall sewers. In the larger areas it is approximately equal to 320 gal. per capita daily for a population of 20 per acre.

In designing general water systems the same uncertainty as to the future exists, and the engineer must avoid the extremes of hopeless inadequacy and dangerous optimism. Such a layout as will doubtless ultimately be required would involve capacities greater than are likely to be needed for many years, with consequent unduly heavy present assessments; it is also necessary to avoid the danger of insufficiency of service in too short a period. The possibility of future reinforcing mains and the character of the fire risk are taken into consideration in the Franklin County Department, and the general rule is to design for 20 people per acre, using an average of 50 gal. per capita, with a maximum of 225% of the average, plus a fire provision equal to three fire streams for the first 500 acres, plus one stream for each additional 500 acres. In view of the residential character of the expected development, this should provide satisfactory service for a reasonable period of gradual growth. In case of greater demand, supplementary mains will have to be constructed later.

Assessments

Perhaps the most interesting feature of the work is the apportionment of assessments for trunk sewers and the larger water mains. The law provides

that, where such works are constructed, the abutting property shall be assessed for local service, and the excess cost, less such part as may be paid by the county at large, shall be assessed over the entire area benefited, including the abutting property, as a district assessment divided in proportion to benefits.

The first question arising from this provision is as to the amount, if any, that should be paid by the county, and included in general taxes. In municipal work, it is frequently provided by law that some part of sewer and water systems be financed by such taxes levied over the entire city. J. L. Van Ornum, M. Am. Soc. C. E.,* found that 19 out of 50 cities investigated paid the cost of main sewers by general taxation, and that 6 more paid the cost less a local assessment on abutting property. He states that this is proper in view of the general benefit to the community due to the establishment of a proper sewer system. In a report to the Sewerage Commissioners of Brockton, Mass., published in 1894, F. Herbert Snow, M. Am. Soc. C. E., stated that Massachusetts law required that from one-fourth to two-thirds of the cost of common sewers should be paid from tax levies, and that, in his judgment, in the case of a separate system, one-fourth of the total cost of the system fairly represented the share of benefit accruing to the city as a whole.

There is, however, an essential difference between municipal and county conditions, in that all parts of a city may reasonably be expected to require sewers and water mains, either now or in the very near future, while counties include, as a rule, large areas of agricultural land so remote from any city as to remove any possibility of urban development for generations. A general county tax must include these farm lands and, except in rare cases and under very unusual conditions, the payment of any part of the cost of this work by such taxation is unjust and cannot be considered. If the law permitted the levying of a tax within the limits of a sewer district, it would be a very different matter, but as this is not the case and since it is necessary to consider each sewer district as a separate body politic in designing and financing these improvements, the cost of each system must be defrayed by the assessment method.

When the law, as in this case, requires apportionment in proportion to benefits, it is almost universally held that the judgment of the assessing board will not be disturbed by the Courts unless fraud is shown or confiscatory assessments levied. This principle, together with other rules affecting assessments, has been very clearly stated in a recent Ohio decision.†

Assessment Proportional to Enhancement

The writer is of the opinion that enhancement of value is the proper and logical, as well as the legal, measure of benefit, and that the use made of the property at the time should not be considered, except as it may affect the major question. An assessment once levied cannot be changed, and, if he chooses, the owner of vacant land may construct buildings soon after the assessment is levied, that will make greater use of the sewer than those previously built by his neighbors. Failure of any owner to utilize his land should

* "Theory and Practice of Special Assessments," *Transactions*, Am. Soc. C. E., Vol. XXXVIII (1897), p. 336.

† Rogers *vs.* Johnson, Treas., 21 Ohio App. May 7, 1926, Mauck, P. J.

not relieve him of his proper charge. These principles are supported by the best legal authorities and by numerous Courts.*

The enhancement basis being adopted, it is evident that the character and value of the land become factors in estimating benefit. It has been held by the Courts that either valuation or area, or both, may constitute a proper basis for such apportionment. Mr. Snow gives an illustration showing that the valuation basis, if used in direct proportion, must work a hardship on the more valuable property, but it may be safely assumed that land worth $2 000 per acre must appreciate in value (expressed in dollars per acre) to a greater extent than $200 land, when furnished with sewer and water facilities. Assuming that the cost of these improvements was equal to an average of $200 per acre, it cannot be true that the one is benefited only 10%, while the other is doubled in value. The fact doubtless lies somewhere between the two extremes and after much consideration the writer has reached the conclusion that if, in connection with the other factors later described, the assessment is apportioned in the ratio of the square root of the values, substantial justice will be done. This brings about a result approximately equivalent to spreading one-half the assessment in the ratio of value and one-half on a direct area basis. Land values only are considered, as the buildings are under control of the owner and may be entirely changed at any time.

It is admittedly impossible to determine the precise effect of any public improvement on the market value of property in the district. All that can be done is to estimate what should be the effect. All authorities agree that it is impossible to apportion an assessment with absolute accuracy and correctness, but an effort should be made to establish a method which will result in substantial equity.

As far as the writer has been able to learn, no attempt has ever been made to analyze this problem, although Professor Van Ornum refers to a case in which various factors were considered, but without explaining the method used. Many engineers have adopted a more or less arbitrary plan of establishing zones at varying distances from the improvement and fixing percentages representing the relation of the rates in such zones. In many cases, the assessment is apportioned simply in accordance with the personal judgment of the officials as to relative benefits. The plan originated by the writer, which is used in Franklin County, and adopted by several other counties in Ohio, represents an effort to establish a fair and equitable basis and, as it is believed to present some new ideas, will be described in detail.

Method of Apportionment

The plan is based on the following premises:

1.—The measure of benefit is the difference in value of the property before and after the improvement is made.

2.—Valuable land is benefited more than cheap land, although not in proportion to the ratio of the values, the square root of the value ratio being assumed to represent this item fairly.

* City of Butte 2, School Dist. No. 1, 29 Mont. 336 339, 74 Pac. 869 (1904). See, also, "Taxation by Special Assessments," by Page and Jones; "Law of Special Assessments," by Hamilton; and "Municipal Corporations," by McQuillan.

3.—The present use of the land, or the presence or absence of buildings is not ordinarily a factor, being subject to radical change and wholly under control of the owner.

4.—In the case of a district completely sewered at one operation, the cost of trunk sewers and disposal facilities could fairly be assessed over the district on an area basis, subject only to the valuation correction, and such an apportionment should represent a reasonable estimate of the enhancement of value.

5.—When only the main works of the system are built, designed to serve the whole district ultimately, the assessment should be so adjusted that when the system is completed, each property in the district will have been assessed equally per unit of area, as of the date when service is given, subject to correction for valuation.

6.—Following the thought of Premise 5, it is evident that property which will later be assessed for a share of main sewer extensions or branch mains, is correspondingly less benefited by the sewer now built than property which will not require such an extension or branch. This feature of the problem may be called "accessibility".

7.—It is equally clear that property which is not given immediate service is less benefited than that which has such service, and that a fair measure of this factor, which may be called "time", is the ratio of the present worth of a dollar, for a period equal to the probable time which will elapse before service is rendered, to the basic dollar.

In order to develop such a distribution of the district assessment it is necessary to estimate the cost of future extensions and branch mains, the period at which each portion of the district will be so developed as to require sewerage, and the value of the land. The first is a simple matter of estimating costs from the general plan. The second is obtained by dividing the district map into rather small zones, calling in the services of several qualified real estate experts familiar with the district, securing the estimate of each as to the reasonable development period for each zone, and striking an average representing their combined judgment. The third item was formerly handled in a similar manner, but a recent official re-appraisal of all property in the county by a force of competent appraisers has made this unnecessary.

Computation of Assessments

The detailed procedure in distributing the district assessment for a trunk sewer on these principles is as follows:

1.—Present and Future Excess Cost.—Estimate the total present and future excess cost of all trunk and branch sewers of the system above the part of such cost which will be assessed on abutting property for local service.

2.—Sub-Districts.—Divide the district into sub-districts (for purposes of assessment only), each of which includes the whole area served by an extension or branch larger than 8 in., or which contributes either directly or by 8-in. laterals to the main trunk.

3.—Time Zones.—Divide these sub-districts into the several time zones representing the period when service is expected to be provided, such as "immediate", "2 years", "5 years", etc.

4.—Accessibility.—Apportion the total present and future excess cost to the several sub-districts in proportion to area, and deduct from the amount

charged to each sub-district its own estimated excess future extension or branch main cost.

5.—Time.—Divide the amount thus apportioned to each sub-district among the several time zones therein and multiply the share of each zone by the present worth of one dollar for the period assumed for that zone. Add the results and determine the percentage relation of each to their sum. Apportion the total present district assessment over the zones in the ratio thus established.

6.—Valuation.—It is not practicable to correct each individual property for valuation. Areas of approximately equal unit value are laid out and a fair average is used for each. The apportionment for each area is then computed from the accessibility and time results and multiplied by a factor equal to the square root of the quotient of the valuation per acre of that area divided by the lowest valuation per acre of any area in the district, or,

$$F = \sqrt{\frac{V}{V_{\text{min.}}}} \quad \text{.........................(1)}$$

in which, F is the factor applied to any area, A; V is the valuation per acre in A; and $V_{\text{min.}}$ is the lowest valuation per acre of any area within the district. The percentage relation of each result to their sum is determined, and this percentage, applied to the total amount of the present district assessment, gives the amount to be levied on each valuation area. From this, the rate per acre is readily determined and the apportionment for each property is a simple matter.

In most districts there are both platted and unplatted areas. In order to avoid discrimination against the latter, a reduction of 15 to 20% is made in all unplatted areas to allow for future streets. In platted territory, after fixing the total amount to be collected from each valuation area, this amount is distributed, using frontage, or more correctly, average lot width, as a general guide, this usually appearing to represent an equitable division. In some unusual cases, a combination of frontage and area has been used, but, with ordinary regular platting, this is unnecessary. Occasionally, some local condition requires special consideration; but, in general, assessments can be computed by the method described with satisfactory results.

These computations are not as difficult or complex as they seem. In the case of large districts they require a considerable amount of time and effort, but if they result in a fair and equitable solution of the problem, it is worth while. It may be mentioned that in the practical application of this method, it has been found that the assessments levied against outlying property not now given actual service, approximate reasonably the cost of the additional capacity provided to meet their future needs.

Typical Computations

The process may be made more clear by the following example of an actual assessment in Franklin County, Ohio, covering one of the smaller projects. It will be seen that it results in placing the weight of the assessment on the valuable and immediate property and reducing the burden on the cheap and distant land to a small figure.

Computation of Assessment Rates; Marion Road Sewer.

Amount to be assessed, $11 607.63.

Total present and future excess cost, $14 338.26.

Total area to be assessed, 622.85 acres.

TABLE 1.—ACCESSIBILITY.

Sub-district. (1)	Acres. (2)	Average rate per acre. (3)	Preliminary apportionment. (4)	Future excess. (5)	Adjusted preliminary apportionment. (6)
A	240.47	$23.02	$5 535.72	$1 785.33	$3 750.39
B	84.12	23.02	1 936.48	224.10	1 712.38
C	60.79	23.02	1 399.41	250.80	1 148.61
D	38.10	23.02	877.08	268.80	608.28
E	61.51	23.02	1 415.98	201.60	1 214.38
F	137.86	23.02	3 173.59	0.00	3 173.59
Total......	622.85	......	$14 338.26	$2 730.63	$11 607.63

TABLE 2.—TIME.

Zone. (1)	Time, in years. (2)	Zone area, in acres. (3)	Sub-district area, in acres. (4)	Adjusted preliminary apportionment for sub-district. (5)	Present worth of one dollar. (6)	Present worth. (7)	Percentage of total. (8)	Time adjustment of excess cost. (9)
A 1	5	26.05	240.47	$3 750.39	$0.747	$303.49	4.13	$479.40
A 2	10	49.80	240.47	3 750.39	0.558	433.39	5.90	684.85
A 3	15	71.08	240.47	3 750.39	0.417	462.27	6.29	730.12
A 4	20	93.54	240.47	3 750.39	0.312	455.16	0.20	719.67
B 1	5	4.68	84.12	1 712.38	0.747	71.16	0.97	112.59
B 2	8	3.57	84.12	1 712.38	0.627	45.56	0.62	71.97
B 3	15	43.21	84.12	1 712.38	0.417	366.79	4.99	579.22
B 4	20	32.66	84.12	1 712.38	0.312	207.42	2.82	327.34
C 1	8	3.57	60.79	1 148.61	0.627	42.29	0.58	67.32
C 2	15	42.19	60.79	1 148.61	0.417	332.41	4.52	524.66
C 3	20	15.03	60.79	1 148.61	0.312	88.60	1.21	140.45
D 1	3	14.89	38.10	608.28	0.84	199.69	2.72	315.73
D 2	7	23.21	38.10	608.28	0.665	246.42	3.36	390.02
E 1	3	21.95	61.51	1 214.38	0.84	364.02	4.96	575.74
E 2	5	20.34	61.51	1 214.38	0.747	299.97	4.08	473.59
E 3	7	19.22	61.51	1 214.38	0.665	222.34	3.44	399.30
F	Immediate	137.86	137.86	3 173.59	1.00	3 173.59	43.21	5 015.66
Totals....	....	622.85	....	....	....	$7 344.57	100.00	$11 607.63

EXPLANATION OF TABLES

Table 1 contains the computations to determine the preliminary apportionment of costs on the basis of accessibility. The average rate per acre (present and future), is found to be $\frac{14\,338.26}{622.85} = 23.02$. The values in Column (4) are the products of Columns (2) and (3), and show the amount

which would have been chargeable to each sub-district (subject to correction for valuation) if the complete system had been constructed at this time. Column (5) shows the estimated excess cost of the future mains in each sub-district. Sub-District F is the area of present service. Column (6) is compiled by subtracting Column (5) from Column (4) for each sub-district, crediting each sub-district with its future expense.

TABLE 3.—VALUATION.

Area. (1)	Value, per acre, (V). (2)	Area, in acres. (3)	Total area of time zone, in acres. (4)	Adjustment of excess cost for time zone. (5)	Value of $F = \sqrt{\frac{V}{200}}$. 6)	Time assessment $\times F$. (7)	Percentage of total. (8)	Value adjusted for excess cost. (9)
A1(*a*)	$ 500	23.13	26.05	$ 479.40	$1.581	$ 672.97	2.92	$ 338.93
A1(*b*)	750	2.92	26.05	479.40	1.936	104.03	0.45	52.23
A2	300	49.80	49.80	684.85	1.225	838.94	3.64	422.52
A3	300	71.08	71.08	730.12	1.225	894.40	3.88	450.38
A4	200	93.54	93.54	719.67	1.	719.67	3.12	362.16
B1	500	4.68	4.68	112.59	1.581	178.00	0.77	89.38
B2	800	3.57	3.57	71.97	2.	143.94	0.62	71.97
B3	300	43.21	43.21	579.22	1.225	709.54	3.08	357.51
B4	200	32.66	32.66	327.34	1.	327.34	1.42	164.83
C1	800	3.57	3.57	67.32	2.	134.64	0.58	67.32
C2(*a*)	300	31.31	42.19	524.66	1.225	476.96	2.07	240.28
C2(*b*)	400	8.16	42.19	524.66	1.414	143.48	0.62	71.97
C2(*c*)	800	2.72	42.19	524.66	2.	67.64	0.29	33.66
C3	200	15.03	15.03	140.45	1.	140.45	0.61	70.81
D1	750	14.89	14.89	315.73	1.936	611.25	2.65	307.60
D2	400	23.21	23.21	390.02	1.414	551.49	2.39	277.42
E1	750	21.95	21.95	575.74	1.936	1 114.63	4.83	560.65
E2	600	20.34	20.34	473.59	1.732	820.26	3.56	413.23
E3	400	19.22	19.22	399.30	1.414	564.61	2.45	284.39
F1	1 000	27.00	137.86	5 015.66	2.236	2 196.46	9.52	1 105.05
F2	1 250	25.24	137.86	5 015.66	2.5	2 295.72	9.95	1 154.96
F3	1 500	20.04	137.86	5 015.66	2.739	1 997.01	8.66	1 005.22
F4	1 750	24.32	137.86	5 015.66	2.958	2 617.29	11.35	1 317.47
F5	2 000	41.26	137.86	5 015.66	3.162	4 746.58	20.57	2 387.69
Totals	....	622.85	....	....	....	$23 067.30	100.00	$11 607.63

The effect of time in the adjustment of excess costs is computed as shown in Table 2. Column (1) designates the division of Sub-Districts A, B, C, etc., into zones that will feel the benefits of the improvement in various estimated periods of time. The corresponding time intervals are listed in Column (2). Column (3) lists the area of each zone. Columns (4) and (5) correspond to Columns (2) and (6) in Table 1. Column (6) gives the present worth of $1.00 at the end of the time interval indicated. Column (3) divided by Column (4) and the result multiplied by Column (5) gives the adjusted preliminary apportionment for each zone, and this multiplied by Column (6) gives the results shown in Column (7), or,

$$\text{Column (7)} = \frac{\text{Column (3)}}{\text{Column (4)}} \times \text{Column (5)} \times \text{Column (6)}$$

The total of Column (7) is, of course, less than the amount to be assessed and it is, therefore, proportionally expanded to the correct amount as indicated in Columns (8) and (9).

Finally, the computations for the determination of the valuation correction are given in Table 3. It will be noted that some of the zones in Column

(1) have been further subdivided in order to allow for a difference in valuation. In Column (2) are listed the values of the various areas per acre. The acreage of each valuation area is given in Column (3) and, for convenience in computation, Column (3), Table 2, is repeated as Column (4). Column (5) is a duplication of Column (9), Table 2. In Column (6) are listed the values of the factor, F, which equals $\sqrt{\frac{\text{Column (2)}}{200}}$. Column (3) divided by Column (4) and the result multiplied by Column (5) gives the time assessment for each valuation area and this multiplied by Column (6) makes a preliminary correction for value, shown in Column (7), or,

$$\text{Column (7)} = \frac{\text{Column (3)}}{\text{Column (4)}} \times \text{Column (5)} \times \text{Column (6)}$$

The total of Column (7) being greater than the amount to be collected, the items are reduced proportionally as indicated in Columns (8) and (9). Column (9) of Table 3 gives the amounts to be assessed in each area, and all that remains is to distribute them over the individual properties within the areas, the rate per acre being fixed by dividing each amount by the number of acres in its particular area, or,

$$\text{Rate per acre} = \frac{\text{Column (9)}}{\text{Column (3)}}$$

Storm Sewer Assessments

In the case of storm sewers special consideration must be given to land relieved of overflow conditions or otherwise benefited to a greater degree than other property, but if this feature is taken care of in a manner similar to local service cost as described in the detailed procedure previously outlined, the same methods can be thereafter used.

Assessments for Water Mains

Water mains present a different problem in one regard, that is, that the boundaries of the district to be served are not fixed as in the case of a sewer. It has been found that the only practicable and satisfactory method is to lay out a definite area on both sides of each main, covering a reasonable zone of influence with reference to other present or future mains, and ignore the inevitable circulation and overlapping of service through the several feeders, assuming the benefit of each to extend to a line between such feeders at a distance fairly representing the proportionate capacity. This being determined, the regular method may be used.

Conclusion

The system described has been in use for several years. In general, it has seemed to meet with public approval, and no difficulty has been found in explaining it in simple language to the average property owner. Many expressions of approval of the principles involved have been received from such owners as well as from attorneys and engineers. It is presented for consideration and discussion as an attempt to solve, as far as may be, a problem which has apparently heretofore received less attention than its importance merits.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE PRACTICAL UTILITY OF HIGHWAY TRANSPORT SURVEYS*

By G. F. Schlesinger,† M. Am. Soc. C. E.

Synopsis

This paper describes how the data obtained in a State-wide traffic and transport survey of the State highways of Ohio were utilized in formulating a plan and budget for future highway improvements. The field information was collected and recorded during a period of one year—December, 1924, to December, 1925. The routes of the State system of highways were classified as to traffic importance, based on the following considerations: (1) Density and character of traffic applied, for convenience of analysis, to five sections of the State; (2) maximum concentration periods; (3) influence of truck and bus traffic; (4) influence of population; (5) traffic service afforded by continuous routes and required by the needs of the individual citizen; and (6) the development and growth of future traffic. The plan of improvement was based on the traffic classification and the existing condition of the highways. A budget of the estimated cost to complete the plan in a 5-year period was prepared.

Procedure for Ohio Survey

A highway transport survey is an accurate and comprehensive collection of traffic data, followed by study and analysis. A number of States, cities, and other political subdivisions have carried on so-called traffic surveys with the object of securing information that would be of value in the solution of their highway problems. The Federal Government, through its Bureau of Public Roads, has taken a leading part in similar investigations. Prior to 1924, it had co-operated with the Highway Departments of California, Connecticut,

Note.—Written discussion on this paper will be closed in January, 1929.

* Presented at the meeting of the Highway Division, Columbus, Ohio, October 13, 1927.

† Chf. Engr. and Managing Director of the National Paving Brick Mfrs. Assoc., Chicago, Ill.

Maine, and Pennsylvania, and also had made an intensive study of traffic in Cook County, Illinois.

A recent project of this kind was a survey of transportation on the State, county, and township road systems of Ohio under a co-operative agreement between the Bureau of Public Roads and the Ohio Department of Highways and Public Works.* Due to the activities of the Bureau, the methods of making such surveys on rural highways have approached standardization. The Ohio survey was begun in December, 1924, and continued for a period of a year, thus covering all seasonal variations. Data were recorded at 1 158 points located so as to obtain the variations in traffic of various routes and sections of routes. Stations were also located on representative secondary county and township roads. At 358 of these points complete data were recorded one day each month during the year period. At the remaining 800 points counts of passenger cars and motor trucks were obtained on three days during the summer months. Data obtained at these stations included a count of passenger cars, motor trucks, motor buses, horse-drawn vehicles, foreign vehicles, and detailed truck and passenger-car statistics. Motor-truck data included the capacity of the truck, State of registration, place of ownership, origin, destination, type of origin and destination, commodity carried, and tire equipment. For alternate periods, at 156 stations, total gross and rear-axle weights were measured by portable scales. Passenger-car data included the State of registration, place of ownership, purpose of trip, origin, destination, and number of passengers.

Each operation consisted of a 10-hour observation period, alternating between 6:00 A. M. to 4:00 P. M. and 10:00 A. M. to 8:00 P. M. Special observers tabulated traffic between 8:00 P. M. and 6:00 A. M. at selected stations, at which, therefore, complete 24-hour observations were obtained. These were made the basis of computation of hourly variations in traffic and of average daily traffic at all stations. Traffic observations for periods of a week were also made at selected stations to determine variations by days. Seasonal changes were computed from the monthly operations at all stations. A carefully planned schedule covered the various days of the week and prevented duplicate recording of traffic.

Information to Be Obtained

The business of the highway departments is to produce transportation service for the users of the highways. A successful head of a business must have full knowledge of his market, that is, reliable information as to the demand for his product, its quantity, and location. A highway transport survey is an effort to supply this information to the engineer executive. Such surveys have been criticized on the grounds that highway authorities cannot

* The report of this survey was published in the fall of 1927. The highway traffic studies upon which it is based, were conducted under the joint supervision of Thomas H. MacDonald, Chief of the Bureau of Public Roads, U. S. Department of Agriculture, and the writer, then Director, and L. A. Boulay, former Director of the Ohio State Department of Highways and Public Works. J. Gordon McKay, Chief of the Division of Highway Economics, Bureau of Public Roads, directed the work of the survey and preparation of the report, assisted by Messrs. O. M. Elvehjem, E. T. Stein, L. E. Peabody and B. P. Root, all of the Division of Highway Economics, and Messrs. Harry J. Kirk, Ohio State Highway Engineer, and Harry E. Neal, Ohio Traffic Engineer.

make practical use of the data obtained; that the survey is interesting as a speculative thesis, but has no utilitarian value. The principal application of the evidence produced is as the basis of a plan of future highway improvement for a period of years. It is logical that such a plan, evolved to meet the requirements of traffic, should be based on the quantity and character of the traffic that it is designed to serve. This type of development from the Ohio survey will be outlined.

The density and character of traffic that existed in 1925 were determined in the field as described. The expected future increase was estimated, and the routes of the State system were classified as to their traffic importance. On this classification was based the plan of improvement for the following 5-year period.

The term, "density of traffic," means the number of motor vehicles passing any given point on a highway in a unit of time. The accuracy of determining it is influenced by the distance between the survey stations. Exactness of method would require a density record for each point on the highway system at which traffic varies. The cost involved, in proportion to the relatively small gain in accuracy, does not justify such a close location of observation points. The density computed for each station on the Ohio Highway System is applied to the short sections of highway that are reasonably adjacent, on which the traffic varies but little.

Variable Highway Conditions

The State highway system of Ohio consists of 11 000 miles of the total of 84 884 miles of rural highways. It is estimated that in 1925 there was a motor-vehicle movement of approximately 3 746 000 000 vehicle-miles on all rural highways and 2 160 000 000 vehicle-miles on the State system. In other words, 13% of the mileage carried 58% of the total motor-vehicle traffic. On the basis of vehicle-miles per mile of highway, the traffic on the State system is more than nine times as intense as that on the secondary rural highways.

The daily volume of traffic on different parts of the State highway system varies widely, as is shown graphically in Fig. 1. Of the 11 000 miles of the State highway system, 131 miles, or 1.2% of the total mileage, carried at the rate of 2 500 or more motor vehicles per day in 1925; 858 miles (7.8% of the system), carried 1 500 or more vehicles per day; 3 239 miles (approximately 30% of the total), carried 600 or more vehicles per day; and 7 761 miles (70.6%), carried less than 600 vehicles per day, of which 4 180 miles carried less than 200 vehicles per day. These relations are shown in Fig. 2.

A basic traffic map of the State was prepared. This is somewhat similar to Fig. 1 and shows the details of distribution of passenger-car and motor-truck traffic, the estimated 1930 traffic, the traffic classification, and the density and trend of population by townships. In its preparation, allowance was made for routes under construction and in poor condition at the time of the survey.

Density of Traffic

For convenience of analysis the State has been arbitrarily divided along county lines into five traffic sections showing distinctive characteristics as

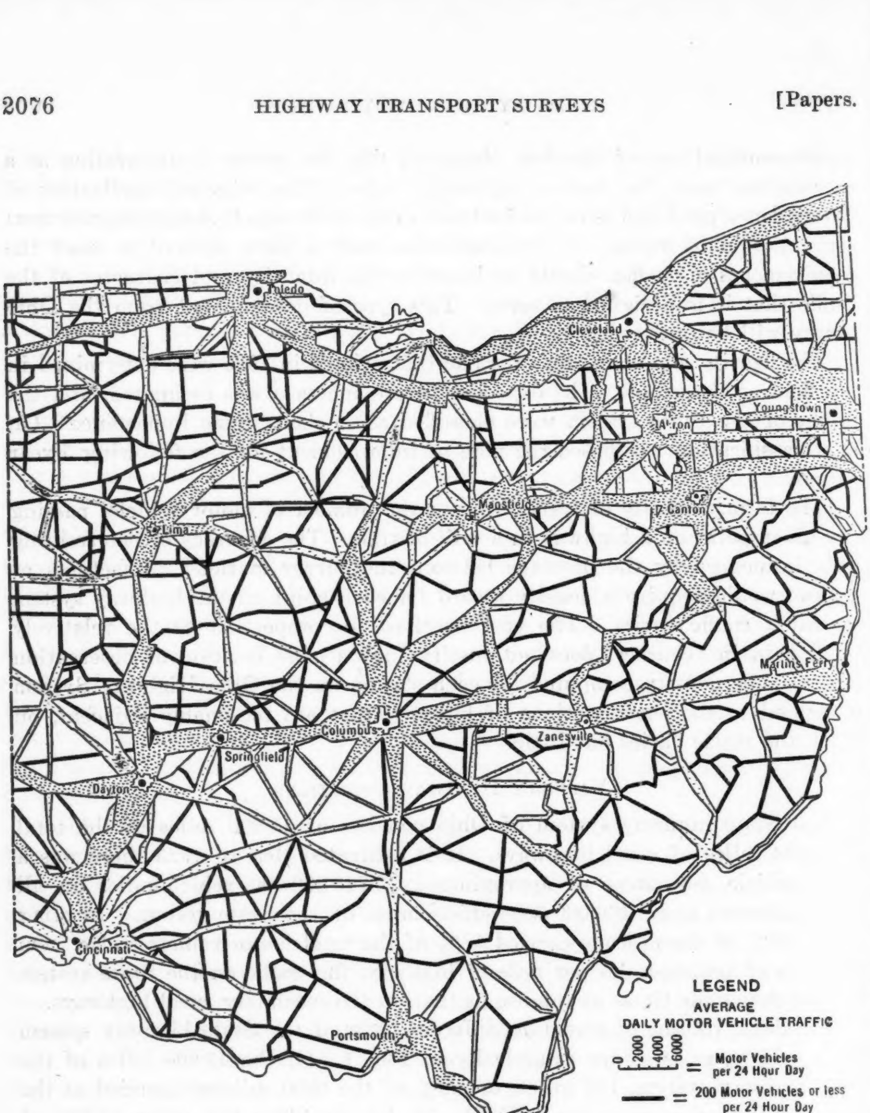

FIG. 1.—MOTOR VEHICLE TRAFFIC ON OHIO HIGHWAY SYSTEM.

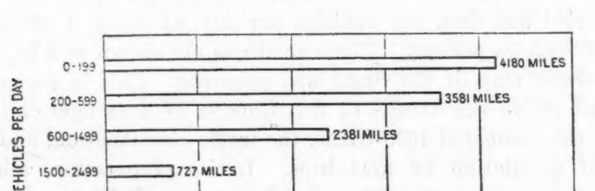

FIG. 2.—MILEAGE OF STATE HIGHWAYS BY TRAFFIC DENSITY CLASSES.

to traffic, population, topography, and industrial development, each of these sections being subdivided in the order of its traffic importance, into two or more divisions, as shown in Fig. 3, which if compared with Fig. 1 will show the location of the traffic sections with reference to traffic density, population, and industry. The relative traffic importance of the sections is shown in Table 1, wherein the mileage of highways in the five sections is classified according to the density of traffic in 1925.

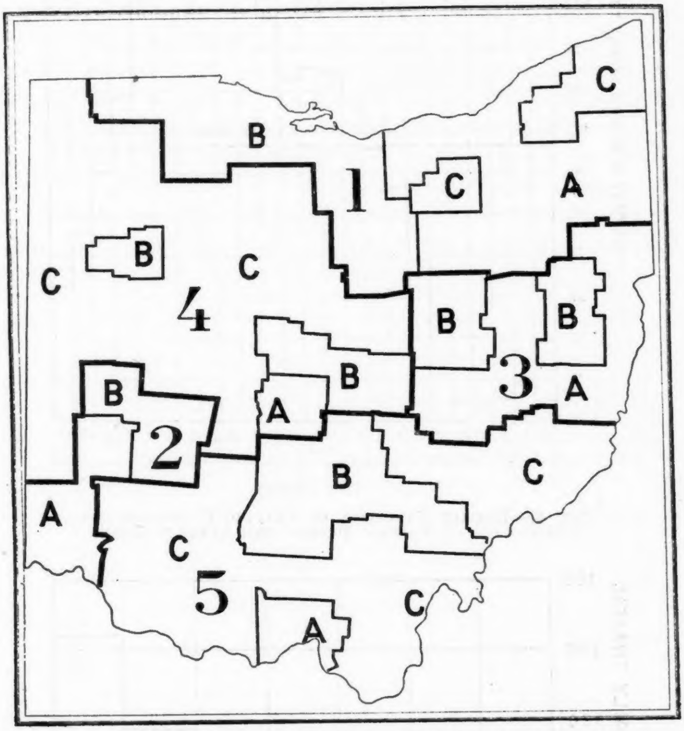

FIG. 3.—OHIO TRAFFIC SECTIONS AND THEIR SUB-DIVISIONS.

TABLE 1.—MILEAGE OF OHIO STATE HIGHWAYS BY TRAFFIC CLASSES AND TRAFFIC SECTIONS (FIG. 3).

SECTION.		ALL STATE HIGHWAYS.		DAILY TRAFFIC, MORE THAN 1 500 VEHICLES.		DAILY TRAFFIC, 600 to 1 500 VEHICLES.		DAILY TRAFFIC, LESS THAN 600 VEHICLES.	
Number.	Location.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.
1.....	Northeastern.......	2 821	25.6	454	52.9	836	35.1	1 536	19.8
2.....	Southwestern......	736	6.7	133	15.5	202	8.5	401	5.2
3.....	East-central........	1 285	11.7	86	10.0	265	11.1	934	12.0
4.....	Northwestern......	3 402	30.9	162	18.9	679	28.5	2 556	32.9
5.....	Southern...........	2 756	25.1	23	2.7	399	16.8	2 334	30.1
......	State total........	11 000	100.0	858	100.0	2 381	100.0	7 761	100.0

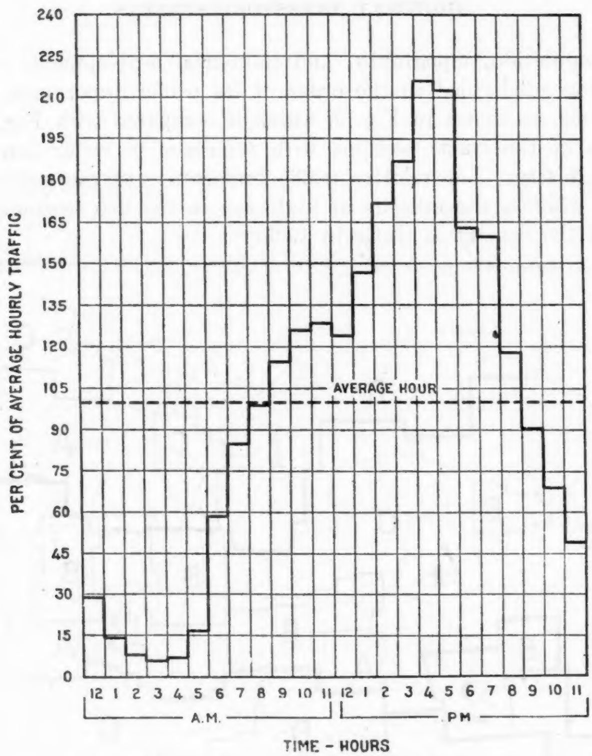

FIG. 4.—HOURLY VARIATION OF TRAFFIC EXPRESSED AS PERCENTAGE OF TRAFFIC DURING THE AVERAGE HOUR.

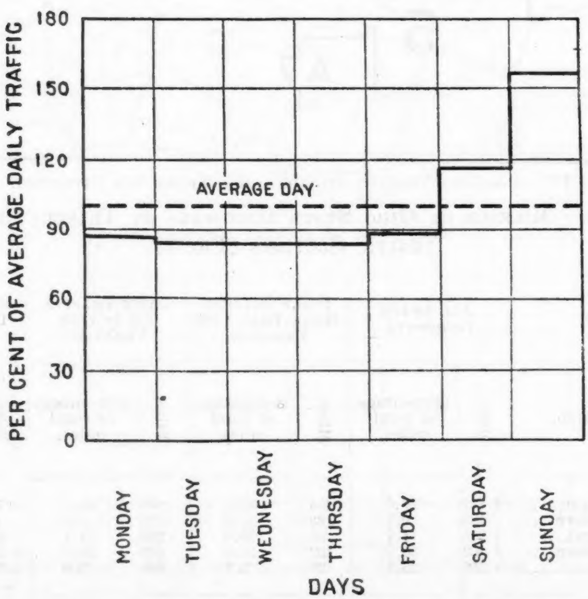

FIG. 5.—DAILY VARIATION OF TRAFFIC EXPRESSED AS PERCENTAGE OF TRAFFIC DURING THE AVERAGE DAY.

Within the northeastern and the southwestern sections (Nos. 1 and 5) the routes carrying 600 or more vehicles per day include almost one-half the mileage. In the northwestern section, also, similar routes serve to connect the more important cities, as well as to form a series of short sections radiating from these centers. More than two-thirds of the State highway mileage in this section, however, has a daily traffic of less than 600 vehicles.

In the southern area, sections of highways carrying from 600 to 1 500 vehicles per day comprise only 15.0% of the State highway mileage in the area; and this relatively small percentage serves the more important cities.

The traffic density for an average 24-hour day by no means represents the peak concentration of traffic. Traffic varies with the season, month, day of the week, and hour of the day. As shown by the survey (Figs. 4, 5, and 6) the normal maximum traffic on a typical road in Ohio would occur on a Sunday in August at 4:00 P. M. By combining the data on these graphs this is found to be approximately 480% of the annual daily average. In the vicinity of the larger cities the maximum concentration of traffic necessitates consideration of multiple-lane surfaces and parallel routes.

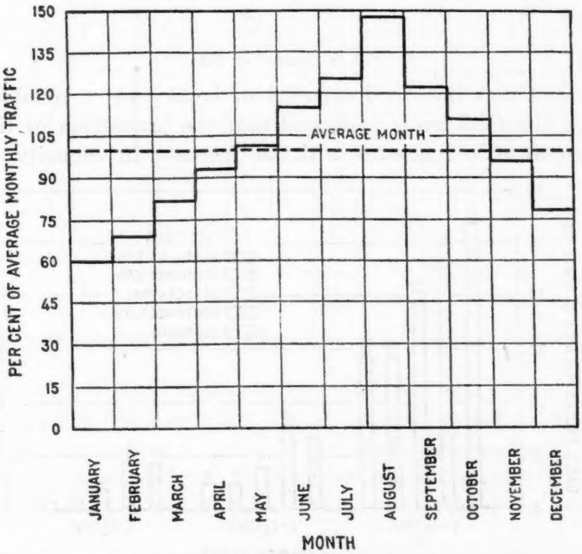

FIG. 6.—MONTHLY VARIATION OF TRAFFIC EXPRESSED AS PERCENTAGE OF TRAFFIC DURING THE AVERAGE MONTH.

INFLUENCE OF TRUCK TRAFFIC

The density and character of motor-truck traffic, as distinguished from passenger-car traffic, must be known in the planning of a system as well as in the design of pavements. It was found to vary greatly on different routes and in various parts of the State. While it averages only 9.5% of the total on the State system, its importance is due to the greater load concentrations, the use of solid tires (15% of all trucks), and a lack of refinement in springs and shock-absorbing parts.

The relative importance of the five traffic sections from the standpoint of truck traffic is shown in Table 2 and Fig. 7. The influence of large cities on density of truck traffic is shown in the two most important trucking areas, the northeastern and southwestern sections, which have five of the seven cities between 30 000 and 100 000 population.

TABLE 2.—TRUCK AND POPULATION DATA IN OHIO.

Section.	Average truck traffic density.	Percentage of total area in the State.	Truck registration per square mile (1924).	Cities of More Than 10 000 Population by Population Classes.*				
				Total.		10 000 to 30 000.	30 000 to 100 000.	More than 100 000.
				Number.	Percentage			
Northeastern....	77	24	7.9	22	44	15	3	4
Southwestern....	75	6	10.9	7	14	3	2	2
East-central.....	53	12	2.5	9	18	9	....	....
Northwestern....	36	30	2.3	7	14	5	1	1
Southern.........	36	28	1.3	5	10	4	1	....
Total........	51	100	3.9	50	100	36	7	7

* U. S. Census of 1920.

It was found that the rated capacity of trucks was a reliable criterion of the weight of the total gross load, and that the proportion of the total gross load on the rear axle increases with an increase in capacity of the truck.

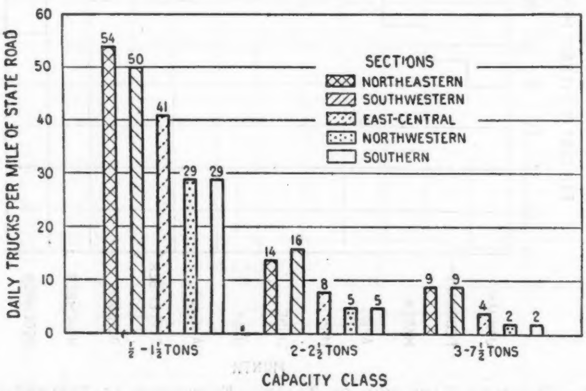

FIG. 7.—DISTRIBUTION OF MOTOR TRUCKS BY CAPACITY CLASSES IN THE FIVE TRAFFIC SECTIONS OF OHIO.

The highway engineer, therefore, must provide for increased load concentrations on routes where large capacity truck traffic is a factor. Only about 2.1% of the trucks with a rated capacity greater than 5 tons were operating on the highways (see Fig. 8). This was evidently due to Ohio's legal maximum gross load limitation of 20 000 lb. (10 tons). It is also in line with the general tendency to prefer medium-sized units as the highway transportation vehicle. It was found that, in general, the heavier gross loads occurred on routes carrying a high density of truck traffic, although there were excep-

tions to this general rule accounted for largely by special movements, such as the hauling of gravel, sand, and stone for construction work.

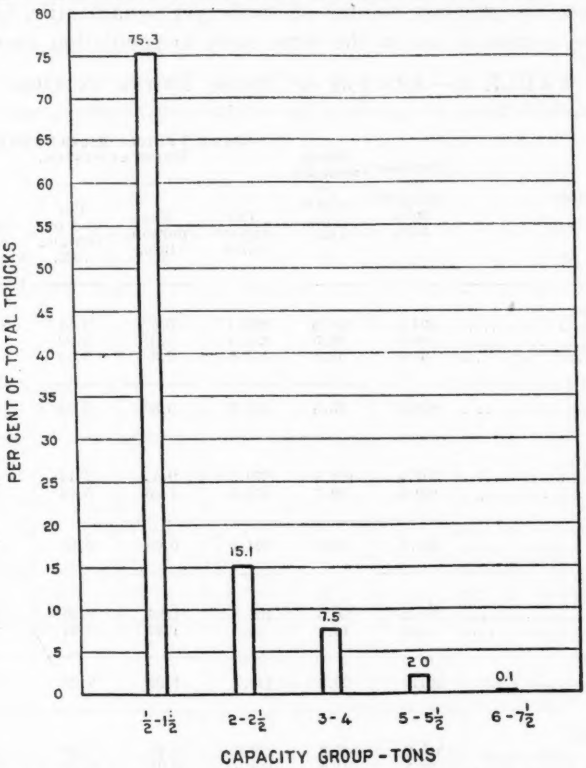

FIG. 8.—DISTRIBUTION OF MOTOR TRUCKS BY CAPACITY GROUPS.

Motor-bus traffic was noticeable. It is relatively the most rapidly increasing type of highway transportation, being largely limited to the State highway system and amounting to 1.5% of the passenger-car traffic during 1925. This traffic must be considered in highway planning, necessitating a high type surface of greater width than would be required by other types of traffic.

HIGHWAY TRAFFIC AND POPULATION

The volume of highway traffic in an area is largely a function of the population of the area. A study of traffic and population shows, that the concentration of traffic is greatest in the areas tributary to the large cities of the State.

Analysis of the trip-mileage of motor vehicles in Ohio shows that 60.4% of the cars observed, travel less than 30 miles per trip, and approximately 70.0%, less than 50 miles; also, that 71.6% of the trucks travel less than 30 miles, and only 14.0%, more than 50 miles per trip. Highway traffic is, therefore, primarily a method of local transportation.

A comparison of the factors producing traffic in the five traffic sections is shown in Table 3. The three divisions having the greatest density of population also have the greatest volume of traffic per square mile, but the traffic on the State system is not in the same ratio to population or motor-vehicle

TABLE 3.—ANALYSIS OF MOTOR TRAFFIC IN OHIO.

Section.	Persons per square mile, 1920.	Motor vehicles per square mile, 1925.	Daily Vehicle Miles on State Highway System.				Persons* per motor vehicle.
			Per square mile.	Per person, 1920.	Per motor vehicle, 1925.	Per mile of State highway.	
I.—Northeastern:							
A	434.5	101.8	292.4	0.67	2.87	1 000	5.13
B	149.5	38.3	204.3	1.37	5.33	684	4.28
C	67.8	18.1	157.6	2.32	8.70	549	3.92
Total	264.1	63.5	236.5	0.89	3.72	804	4.87
II.—Southwestern:							
A	601.4	126.5	270.2	0.45	2.14	944	5.08
B	130.4	30.7	175.6	1.35	5.72	601	4.53
Total	373.7	80.2	224.4	0.60	2.80	776	4.98
III.—East-Central:							
A	131.2	26.5	165.8	1.26	6.26	614	5.25
B	46.6	10.4	66.9	1.43	6.34	281	4.50
Total	101.2	20.8	130.7	1.29	6.29	506	5.11
IV.—Northwestern:							
A	549.2	145.3	259.2	0.47	1.78	1 144	4.25
B	99.1	24.6	199.3	2.01	8.12	723	4.22
C	62.6	15.3	101.0	1.61	6.61	363	4.14
Total	87.5	21.9	119.7	1.37	5.48	434	4.18
V.—Southern:							
A	100.9	24.4	157.3	1.56	6.44	801	4.67
B	71.2	15.2	100.0	1.40	6.56	446	4.75
C	55.0	10.4	74.0	1.34	7.09	291	5.29
Total	61.2	12.3	84.4	1.38	6.87	345	5.07
State total	141.4	32.5	145.3	1.03	4.46	538	4.79

* Based on estimated population, 1925.

registration as in the other sections. This variation is due principally to the following causes:

(1) In the populous sections a larger proportion of the traffic is on city streets instead of on State highways.

(2) In the populous sections the roads other than those on the State system carry a greater proportion of the traffic.

(3) Traffic originating in the populous sections uses highways in other sections to a greater extent.

A county or area should not be required to bear a major portion of the financing of the improvement of a through route for the benefit of traffic that is not of local origin. The distribution of population affects the planning of highway improvements in several important particulars. Densely populated areas require pavements of a width and type that will serve adequately and expeditiously the large volume of traffic, including trucks and buses, between important centers of population. On such routes, separation of grades at railroad crossings, reduction of heavy grades, revision of poor alignment, elimination of curves around parks in small villages, construction of multiple-lane and parted pavements, and the avoidance of traffic deterrents in general are economically justifiable. An adequate plan of highway facilities in such areas requires the provision of "by-pass" routes around cities, which not only are of benefit to traffic on the main routes, but also decrease traffic on the city streets by drivers unacquainted with local conditions.

Although a rigid application of the evidence does not justify the inclusion of as large a mileage of through highways in the less populated sections, the principle of traffic service makes a connected system of routes in such areas a proper function of the State. As shown in Table 3, the vehicle-miles on the State system per person and per motor vehicle in such areas are greater. It would follow, therefore, that the State routes are of greater service to each individual resident of these sections. It is true, however, that highways of width in excess of a normal two-lane road are rarely required and low-type surfaces of the traffic-bound type will give ample service for years in the future.

Forecast of Future Traffic

Any sound plan for highway improvement should take into consideration the traffic of the future in so far as it can be predicted with reasonable accuracy. Ohio had no historical series of traffic counts previous to 1925. From a study of such records of varying length and accuracy, from Maine, Maryland, Massachusetts, Michigan, and Wisconsin, it is evident that in these States, highway traffic has increased directly with the increase in motor-vehicle registration. This assumption was adopted for the Ohio forecast. The registration for a 10-year period until 1935 was estimated by computing future population, using the Bureau of Census method and by extending the persons-per-car curve, the former, of course, having an upward and the latter, a downward trend. By combining the estimated population and the estimated number of persons per car the predicted registration for each year is obtained. The results are shown in Table 4.

In forecasting traffic in 1930 at each traffic station, the procedure was similar to that for predicting motor-vehicle registration in each county, the county rate being applied to each traffic station in the county. For a number of reasons such refinement was not considered justifiable in the forecast for 1935 which was computed on the basis of the State increase in registration rather than on that of the increase in each county.

The methods described will not reflect certain changes in suburban and industrial growth that may occur and influence traffic on short sections of highways. There are other conditions that cannot be anticipated and taken into account. The Ohio Department of Highways is continuing the traffic survey with a skeleton organization, that is, gathering data at certain key stations of the original survey. This work was inaugurated in December, 1926, two years after the original survey was begun.

TABLE 4.—COMPARISON AND PREDICTION OF POPULATION AND NUMBER OF MOTOR VEHICLES IN OHIO.

Year.	Registration (Thousands).		Population* (thousands).	Persons per Car.	
	Actual.	Estimated.		Actual.	Estimated.
1913	86	86	5 095	59.24	59.24
1914	123	127	5 197	42.60	40.92
1915	181	179	5 299	29.28	29.60
1916	252	245	5 402	21.44	22.05
1917	347	324	5 504	15.86	16.99
1918	413	416	5 606	13.57	13.48
1919	511	521	5 708	11.17	10.96
1920	621	637	5 810	9.36	9.12
1921	721	763	5 913	8.20	7.75
1922	859	897	6 015	7.00	6.71
1923	1 069	1 038	6 117	5.72	5.89
1924	1 242	1 181	6 219	5.01	5.27
1925	1 346†	1 329	6 321	4.70	4.76
1926	1 480†	1 475	6 424	......	4.36
1927	......	1 621	6 526	......	4.03
1928	......	1 763	6 628	......	3.76
1929	......	1 902	6 730	......	3.54
1930	......	2 035	6 833	......	3.36
1935	......	2 607	7 344	......	2.82

* Population as of July 1, of each year.

† Data not available when forecast was made. Estimate differs by 1.3% from actual value in 1925, and by 0.3% in 1926.

TRAFFIC CLASSIFICATION OF STATE HIGHWAYS

Having determined the present and future traffic on the State system of highways the next step in formulating a plan of highway improvement was to classify the routes as to their traffic importance. A highway should be designed and constructed so that it will economically serve present and future traffic. The principle of stage, or progressive, construction contemplates the improvement of the roadway with a surfacing that will be increased in strength as the traffic grows. Such a policy has been resorted to—and justifiably so—by highway authorities when funds were not available at the outset for the construction of a durable type of pavement. However, in order that the highways included in a proposed program may be neither over-designed nor under-designed, they should be divided into traffic groups.

The State highways of Ohio are classified in three groups designated as major, medium, and minor, according to their average daily traffic. Highways classed as "major" include, in addition to the roads carrying 1 500 or more vehicles in 1925, those sections which carried less than 1 500 in 1925, but which are expected to carry 1 500 or more vehicles in 1930; similarly, in 1930, in addition to the sections actually carrying at that time 1 500 or more

vehicles there are included as major traffic highways those which are expected to carry that number in 1935.

A similar method is utilized in classifying the highways of the medium traffic group, which includes sections actually carrying between 600 and 1 500 vehicles per day in 1925 and expected to carry that number in 1930. Minor traffic highways, on the other hand, are those which are expected to carry less than 600 daily vehicles in 1925 and 1930.

The traffic limits selected are in line with general highway engineering practice and were confirmed by a study of maintenance costs on various types of surfacing in Ohio. A traffic density of 600 vehicles per day is the limit at which traffic-bound gravel or stone roads can be economically maintained. For greater intensities this type should be strengthened with bituminous treatment or additional courses. A traffic density greater than 1 500 vehicles per day calls for a standard pavement design—the flexible or rigid types for the lower densities, and usually rigid pavements only for roads carrying large volumes of traffic with the accompanying frequency of heavy loads.

The mileage of major and medium classification in 1930 which is not now improved with surfaces superior to gravel affords a reliable index to the need for new improvements during the next few years. The quantities shown in Table 5 include all highways now having such surfaces, regardless of condition or present width. Included in this mileage is a considerable length in poor condition which must be reconstructed to provide adequate highway service, and a large mileage of narrow surfaces which must be widened in order to serve present and expected future traffic.

TABLE 5.—MILEAGE OF SURFACES SUPERIOR TO GRAVEL IN 1925 AND 1930.

Section.	Mileage of major and medium classification, 1930.	Mileage of major and medium classification improved with surfaces superior to gravel, 1925.	Mileage of major and medium classification not improved with surfaces superior to gravel, 1925.
Northeastern	1 985	1 160	825
Southwestern	469	282	187
East-central	547	323	224
Northwestern	1 370	620	750
Southern	850	303	547
Total	5 221	2 688	2 533

In the east-central, northwestern, and southern sections more than one-half the total State highway mileage is expected to remain in the minor traffic class in 1935.

THE OHIO IMPROVEMENT PLAN

Having determined a traffic classification of the highways on the State system considering traffic needs of the future, the plan of improvement was formulated based on the existing condition of the highways. The mileage of proposed new construction, reconstruction, and widening in the five traffic

sections is shown in Table 6. In the improvement plan for a 5-year period from January 1, 1927, to December 31, 1931 (Table 7), new construction is defined as construction on present unimproved sections of highways and sections where the surface cannot be salvaged because of its condition, location, or alignment. The selection of surfaces for this new construction will be based on traffic and physical conditions. Reconstruction is defined as the rebuilding of worn-out surfaces with the same or a superior surface type. On the other hand, widening is the extension of present surfaces to a minimum of 18 ft. and to a greater width where required. In general, surfaces 16 to 18 ft. in width on minor traffic routes are not included in the widening program. If the condition of the surface requires reconstruction such roads have been included in the reconstruction program. The distinction between widening and reconstruction is not clear in many cases, but for the larger part of the reconstruction mileage the condition of the surfaces is such as to require rebuilding. Where the surfaces are narrow, reconstruction also will include widening of the surface.

TABLE 6.—Proposed Improvement Program.

Section.	Total State highway mileage.	Total Improvement Program.		New Construction.				Reconstruction.		Widening.	
				Surfaces Superior to Gravel.		Traffic Bound.					
		Miles.	Percentage of total miles.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.
Northeastern	2 821	1 511	53.6	295	10.5	167	5.9	496	17.6	553	19.6
Southwestern	736	269	36.6	53	7.2	...	...	88	12.0	128	17.4
East-central	1 285	574	44.7	122	9.5	124	9.6	132	10.3	196	15.3
Northwestern	3 402	1 360	40.0	252	7.4	332	9.8	306	9.0	470	13.8
Southern	2 756	807	29.3	285	10.3	77	2.8	198	7.2	247	9.0
Total	11 000	4 521	41.1	1 007	9.1	700	6.4	1 220	11.1	1 594	14.5

The cost of each of the three classes of improvement was obtained by estimating the changes for grading, minor structures, and the approximate surface type required for the sub-grade conditions and estimated future traffic. Exclusive of bridges and the separation of railroad grade crossings, this will amount to approximately $100 000 000.

The urgent need for widening of present surfaces is indicated by the fact that there were on the State system, in 1925, approximately 4 800 miles of surfaces superior to gravel and less than 18 ft. in width, approximately 1 400 miles being from 10 to 15 ft. wide. On light traffic routes, where the present width is between 16 and 18 ft. and the surface is in good condition, widening can well be deferred to a later period and, therefore, this mileage is not

included in the program. The plan also includes a limited mileage of present 18-ft. pavements, which require additional width to serve existing and expected traffic.

TABLE 7.—PROPOSED OHIO FIVE-YEAR NEW CONSTRUCTION, RECONSTRUCTION, AND WIDENING PROGRAM AND ESTIMATED IMPROVEMENT COST.

Class of improvement.	Miles.		Percentage.	Estimated cost.		Percentage.
New construction		1 707	37.8		$41 122 000	41.2
Surfaces superior to gravel	1 007		....	$34 443 000		....
Traffic-bound	700		....	6 679 000		....
Reconstruction		1 220	27.0		35 188 000	35.2
Widening		1 594	35.2		23 644 000	23.6
Total		4 521	100.0		$99 954 000	100.0

The 5-year new construction, reconstruction, and widening program involves 41.1% of the total State highway mileage. The remaining 58.9% (6.479 miles) consists of surfaces in satisfactory condition and adequate for expected traffic during the next five years.

TABLE 8.—ESTIMATED BUDGET, OHIO STATE HIGHWAY SYSTEM, JANUARY 1, 1927, TO DECEMBER 31, 1931.

Item.	Estimated total cost.	Funds required by State Highway Department.
New construction, reconstruction, and widening of highway surfaces	$100 000 000	$100 000 000
Bridge construction	8 000 000	8 000 000
Railway grade crossing elimination	16 000 000*	8 000 000
Maintenance and repair	53 000 000	53 000 000
Administration	3 000 000	3 000 000
Contingency fund	5 000 000	5 000 000
Total	$185 000 000	$177 000 000

* To be shared equally by the State and railroad companies involved.

The reconstruction and widening program is largest in the heavy traffic sections which contain the largest amount of major traffic mileage. The proposed program for new construction superior to gravel is relatively large in the southern, or lowest traffic, section because of the present small mileage of such types, and it is limited largely to a few important traffic routes.

An estimate of budget requirements for the 5-year period is shown in Table 8. The estimate given in Table 7 did not include such items as the cost of major bridges, railroad grade crossing eliminations, routine maintenance, contingencies, and administration. The elimination of grade crossings presents a serious problem when it is considered that there are more than 1 000 railroad crossings at grade, outside of cities, on the State system.

Conclusion

An endeavor has been made in this paper to describe how the results of a transport survey have been utilized, in a practical manner, by establishing a plan of State highway improvement for a period of years, and preparing a budget required by this plan. Other information made available by a transport survey is useful in many ways, but might be classified as being outside the scope of practicality. For example, in the Ohio survey the composition of passenger-car traffic as to State of registration, touring or non-touring, business or non-business, city or farm ownership, was obtained. The fact that the survey evidence indicated that 87.6% of the passenger-car traffic on the State system was of city ownership had a decided influence on the type of measures adopted by the Legislature of 1926-27 for financing improvements on the State highway system. Other information obtained included such data as: Average mileage per trip for passenger cars and trucks; average number of passengers per car; trucks for hire and privately operated; principal commodities hauled by commercial trucks; classification of commercial and private trucks according to capacity; and a comparison of motor truck and railroad tonnage between selected Ohio cities.

The results of the survey and the plan based thereon were presented to the 1927 General Assembly and were effective in obtaining sizable appropriations for State highway improvements, although not quite as large as called for by the budget (Table 8). The main benefit to be expected from the establishment of the plan is that the Ohio Highway Department now has a definite aim toward which it can direct its future efforts. The fact that the Federal Highway Bureau was a party to its formulation, and that it has been endorsed by the non-political organizations of the State interested in highway progress and the highway industry, would seem to insure its inviolability as the future road program for Ohio.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE ENGINEER'S PART IN MAKING THE HIGHWAY SAFE*

By A. H. Hinkle,† M. Am. Soc. C. E.

It is difficult to draw an exact line as to where the engineer's responsibility ceases with reference to safety on highways. Because the field is rather broad what is said here should not be questioned with reference to the exact limit of responsibility between any two classes of individuals.

Study shows that there are many contributory causes of accidents on highways, although any one accident may have its major source. For example, on a rainy day with the windshield blurred, a driver may have approached, at high speed, a sharp curve having no superelevation, where there were leaves on the wet surface, resulting in his machine skidding off the pavement. It is seen that six different factors may have contributed to the accident. Too frequently the driver of a machine that has been in an accident does not himself realize all the factors. This, in itself, may handicap him in profiting by his past experiences.

For discussion, the causes of accidents may be classified as follows: (1) Incidental to the operator himself, such as bad vision, sleepiness, dull sense, over-work, excessive speed, general disregard for traffic rules, etc.; (2) defects in the automobile, such as imperfect brakes, obscured vision due to poor curtains or large corner posts, glaring or too weak headlights, broken connecting rod, etc.; (3) highway conditions, such as sharp curves, steep grades, slippery surfaces, high crowns, narrow pavements or shoulders, steep ditch slopes, narrow bridges, holes in the surface, lack of superelevation, railroad grade crossing, etc.

The third class of accidents is primarily a problem for the highway engineer. About 30% of all the accidents recorded in Indiana in 1926 were due to highway conditions. It would seem that such statistics would in a measure show the degree of responsibility to be taken by the engineer. While limited finances and other restrictions must of course assume their share of responsibility for the engineer's acts, yet he must offer the proper advice to those to whom he reports. Hence, the engineer is primarily responsible for the

Note.—Written discussion on this paper will be closed in **January, 1929.**

* Presented at the meeting of the Highway Division, Columbus, Ohio, October 13, 1927.

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proper design and upkeep of the highway so as to reduce to a minimum the conditions that contribute to accidents.

Curves

Vision.—A minimum clear sight of 500 ft. between any two points 5 ft. above the road surface seems to have been generally adopted for both horizontal and vertical curves. While this might be recognized as a desirable standard, it should ever be kept in mind that different conditions will affect the safety features connected with the sight distance, so that a 400-ft. clear sight in one location may be no more dangerous than one of 600 ft. in another. For example, if the point in question is at the end of a long tangent in open country, where a high speed is normally invited, a short sight distance becomes far more dangerous than it would if it were closely bounded on either side by curves of lower degree which would normally slow down the speed of a vehicle. Also, in built-up districts where the speed is normally much lower than in open country, a shorter sight distance is permissible.

Horizontal Curves.—In open country where a high speed may be expected, it is desirable from a safety point of view, to limit curves to 5 or 6 degrees. Due to the continually higher speeds at which vehicles travel, it is best where the cost is not great to keep the sharpness of the curve several degrees lower than that considered dangerous at present, thus avoiding what may be a demand in later years to reconstruct on a flatter curve. In mountainous country, and in other strategic locations, it is necessary from an economic point of view to construct much sharper curves; but this may be done in such a way that the danger is not necessarily great. Due to the natural tendency of drivers to reduce speed on a curve, a 20° curve in one location may be no more dangerous than a 6° curve in another. The driver is bound to be more cautious in hilly country, full of curves and grades, than at the end of a long tangent with an unexpected curve.

Perhaps the better way from a safety point of view would be to limit the curve to not more than, say, 6° sharper than any preceding curve ending within 200 ft. of it. From a safety point of view, small curvature, or small changes between adjacent curves, regardless of degree, may not necessarily be dangerous. In fact, due to the greater caution naturally taken by drivers on a curved line, it is possible that with the proper design an irregular line with low degree curves or a low differential may be safer than the average tangent. Statistics of accidents seem to show this to be true. Danger is a factor of speed, other things being equal, and, therefore, anything that contributes to a lower speed may result in fewer accidents.

A precaution should be taken to guard against using the maximum degree of curve where it is not necessary. In many cases where there is a 10° curve, a 5° curve could just as well have been used and even more economically because of the saving in pavement.

Superelevation.—Curves of 2° or more should be superelevated. This generally may be computed from the formula:

$$e = \frac{V^2}{15\,R}$$

in which,

e = the superelevation, in feet per foot of width;
V = the velocity, in miles per hour; and
R = the radius of curve, in feet.

In this formula, the speed is assumed at 30 miles per hour, with the maximum superelevation as 0.1 ft. per ft. of width.

The slight difficulty due to a vehicle slipping toward the inside of a curve on an icy surface, may be overcome by building a comparatively wide inner shoulder at such places at the normal shoulder slope. The outside shoulder for its full width should take the same superelevation slope as the pavement. On superelevated curves the crown of the road should be reduced to zero at the point of maximum superelevation.

Widening on Curves.—It is well to widen the pavement on all curves of 5° or sharper. The widening should be on the inside and the width, in feet, should be about 0.3 times the sharpness of the curve, in degrees.

Due to the added cost, the widening on concrete pavements is frequently omitted on a curve of less than 8 or 10 degrees. This may be justified on light traveled roads. In such cases it is necessary to give more care to the inside shoulder by maintaining an added width of stone or gravel that will prevent a rut from forming along the pavement. As an alternate to widening the curve, a spiraled curve is sometimes used. The spiraling is not a proper substitute, however, for widening on high degree curves.

Widening the intersection of important roads has a great value in reducing accidents, perhaps as much in impressing the driver more emphatically with the fact that he is approaching an important and hence dangerous intersection as in increasing the sight distance or giving more room for dodging the other vehicle. At a certain right-angled intersection, one of the two heavily traveled highways is in a cut for about 100 ft. back. This intersection was widened at all four corners to a 100-ft. radius. Although accidents had been numerous previously, not one has been recorded in the two years since the highway was improved. It is interesting to note how drivers slow down as they observe the widened intersection. Unconsciously, the improvement seems to impress the mind of the driver that there is danger ahead.

Grades

From a safety point of view it is desirable to keep long grades down to 3 or 4%, if no material additional cost is entailed. On main heavily traveled highways it is extremely desirable to limit all long grades to 5% even at considerable expense. On secondary roads, 7% may be taken as a desired maximum for long grades. For short distances, say, up to 300 ft., 7% grades may not necessarily be dangerous on the main highways, nor 9% grades on secondary highways. It should be kept in mind, however, that the same theory of relationship applies to grades as to curves. A steep grade will be much less dangerous if approaching lighter grades and curves have slowed down the speed. Thus, frequently, the steepest grades and sharpest curves are not the greatest source of danger.

Combined Grades and Curves

Care should be taken in combining maximum grades and curves. If each alone is a source of danger, their occurrence on the same stretch of road is sure to increase the hazard. Hence, the combination of the steepest grade and sharpest curve on any road should always be avoided.

It is difficult to determine what compensating value should be assigned to curvature in conjunction with maximum grade. Perhaps it is not far wrong to assume that 5° of curvature may be equivalent to at least 1% of grade. In hilly or mountainous country, it is well to guard against the mistake (frequently made) of securing long uniform grades at the expense of sharp curves.

Crown

The higher the crown the more dangerous is any pavement, especially on a rural highway where high speed prevails. Perhaps the effect is almost negligible up to 2 in. of crown in 18 ft. of width; however, on grades and curves any camber increases the danger to traffic. One should not be misled by the fact that the ordinary gravel and stone surface requires more crown on grades to prevent erosion of the surface and thus make the mistake of applying this same dangerous design to grades on pavements. Sometimes it is possible to carry the water down the slope on the pavement more cheaply than in the side ditches. In these cases the pavement itself should be widened 4 to 6 ft., at the same time narrowing the shoulder to a few feet with no side ditch, all water going down the pavement which should have curbs. This design in deep cuts will often save enough in excavation (to say nothing of future maintenance) to pay for the extra width of pavement. It necessitates spillways at the foot of the grade, but practically obviates the crown.

On a modern high type pavement there seems to be little excuse for using more than 1½ in. of crown in 18 ft. of width. A favorite highway warning sign is, "Dangerous When Wet". While the pavement may be on a light grade and of a type of surface which is more or less slippery when wet, the dangerous feature is greatly increased by the unnecessarily high crown. It may be difficult to prove absolutely the amount of additional danger due to the high crown; however, as there is no economy in such construction and little or no cost in eliminating it, the camber might just as well be kept to a minimum.

Width of Pavement and Road Shoulders

Exactly what effect the width of pavement may have on the safety of a highway is open to argument, because the wider the pavement, the higher the speed; and speed is a factor of danger. From observation, however, it seems that although the wider pavement may invite a higher speed, it affords at the same time a net contribution to the increased safety of the highway.

The accident statistics of the National Road leading east out of Indianapolis, Ind. (a 30-ft. pavement), as compared with those on the 18-ft. pavement going westward out of the same city, are instructive. The traffic on these two roads should be about the same. There seems to be no evidence

proving the claim so frequently made that a three-lane pavement is more dangerous than one of two lanes. Out of the fifteen accidents reported on the 30-ft. pavement all but two happened at intersections due to traffic driving on the wide pavement from the side roads or private drives. No serious injuries or great damage occurred. On the other hand, on the 18-ft. pavement there was one fatality, three pedestrians were seriously injured, and twelve turn-overs or collisions occurred, in which the occupants received minor injuries. These meager records point to greater safety on the three-lane pavement.

Wide shoulders properly maintained serve traffic in an emergency. On the 18-ft. country pavements a 6-ft. shoulder has been used as a standard. On heavily traveled roads an 8-ft. width certainly is desirable, particularly where fills are made with a 1 on 1.5 slope. Not only do these shoulders serve traffic in an emergency avoiding an accident but also as a parking place for changing tires or other reasons. Numerous accidents are traceable to cars parked on country pavements and most States have laws prohibiting such practice. However, unless sufficiently wide shoulders are provided the practice of parking on the pavement will be difficult to control.

Sufficient Width of Right of Way

While heavily traveled roads are requiring wider pavements and wider roadbed, it is also imperative to provide sufficiently wide rights of way. Every tabulation of accidents shows a growing number of collisions with telephone poles and other appurtenances. For any important country road, 60 ft. should be the minimum width of right of way, poles being placed at the outer edge. Heavily traveled highways where unusually wide pavements are necessary, should have the needed additional width. There is no excuse for sitting by and permitting people to be killed by running into obstructions that should have been properly located, not only for safety, but to provide a more artistic appearance and room for roadside planting. That many a heavily traveled road with lines of high poles close to the traveled way looks like an alley, is no credit to the engineer or other official who may be responsible for this condition. Many States have the practice of securing 80 and even 100 ft. as a standard for their main State roads. Such widths are not practical through rich agricultural lands, but in the sparsely settled, low-valued districts, they are commendable.

Slippery Pavement Surfaces

It is well known that a slippery road surface contributes to accidents. One section of wood block pavement (about four squares) was so slippery, due to the method of treatment, that one or more accidents occurred nearly every time the surface was wet; it was not uncommon to find three or four cars piled up after the beginning of a rain. Concrete and bituminous road surfaces are the most common pavements. Of the bituminous type, the bituminous macadam or surface-treated road is the most frequent in the rural districts. This type of road can be made into a less slippery surface by the proper grade and amount of bituminous treatment and of aggregate

used for covering. On steep grades it is particularly desirable to spread a coating of 1-in. or $1\frac{1}{2}$-in. stone during the last hot weather of the season.

Some concrete road surfaces are far more slippery than others. It is extremely desirable that some research work be carried on to determine what might be done to make the best non-skid type of surface for this pavement. A dry mix and proper finish may be used to reduce the slipperiness to a minimum. It is well known that the so-called "harsh mix" of concrete gives the opposite of the slippery surface sometimes found. To what extent this mix can be used without materially lowering the strength of the concrete should be determined, especially for grades where slipperiness is a big factor.

The oil dropped from motors adds greatly to the accident hazard. A wider pavement, by scattering the oil, aids in reducing the slipperiness. A sprinkling of cinders on such a surface also helps greatly.

Ditches and Ditch Slopes Along the Highway

As the wider shoulder serves in an emergency as a turnout, the inner ditch slope, if more flat, may permit a vehicle to go down and come out again without upsetting. If it is too steep it may be the source of many an accident. There seems to be little excuse for making these slopes steeper than 1 on 3.5. In addition, the flatter slopes are more economical to maintain because of lessened erosion and also the reduced cost of mowing the grass and weeds.

In past years deep ditches for drainage have been constructed along highways. One heavily traveled road paralleled by such a ditch for 4 miles has claimed five lives in 5 years. Another stretch (11 miles) shows a property loss to damaged vehicles of \$24 000 per year. The public as well as engineers should protest against the practice of a land owner deepening the road ditch in order to drain his farm.

Railroad Crossings

Approaches to Grade Crossings.—Narrow highway approaches to railroad crossings often contribute to accidents. In one such instance four years after the approaches were widened not a single accident has been recorded, although previous to the widening, accidents were common. It is a psychological fact that if a driver's attention is occupied with one thing he cannot so well give it to another. Apparently, this was the only principal involving danger at this crossing. Some advocate that a rough road surface at a crossing would slow the traffic and thus increase the safety. It is very likely that all the advantages in slowing down the traffic would be more than outweighed by the detraction of a driver's attention from an approaching train.

Acute Angle of Approach.—At one point in Indiana an acute angle between a main highway and a heavily traveled railroad has been the source of many accidents, due to the wheels of vehicles catching in the groove alongside the rail. This angle (about 15°) is entirely too sharp for safety. It is desirable if not too inconsistent with the alignment, to lay out such crossings at a greater angle than 25 degrees.

Center Posts at Grade Crossings.—At this same crossing center warning posts were erected with a view of reducing accidents. These center posts dur-

ing the short time of their use, were the source of many times more accidents than they prevented. While such posts may be safe in cities on wide pavements where high speed is not common, they are hardly safe on a wide rural highway where high speed prevails and cannot be so well controlled. Doubtless they would cause many more accidents than the grade crossing itself. While the arrangement might shift the responsibility on the automobile driver instead of permitting the railroad company to take a share, it would not contribute to the safety of such crossings.

Elimination of Railroad Grade Crossings.—Grade crossings on all heavily traveled highways should be eliminated as rapidly as finances permit, either by relocating the roads or by building the necessary structures. Frequently, relocation is the cheaper and at the same time safer method. All structures are small sources of danger in themselves. As formerly built with sharp turns at the approaches, they served horse-drawn traffic fairly well; but with modern traffic such structures are a source of danger only lesser in degree than the grade crossing.

Bridge Widths

While narrow roads take their toll, narrow bridges on heavily traveled roads are also a menace. Not only that; due to the frequency with which they are hit and damaged by traffic, in the end they may be more expensive than wider ones. On all important roads, structures of about 20-ft. span or less should carry the full width of the roadbed, that is, pavement plus shoulders. With large structures, because of the cost involved, some sacrifice must be made in safety to fit the treasury. It must also be recognized that any large conspicuous structure, such as a bridge, is more readily observed by a driver and far less apt to be hit than a smaller one, and hence does not need such a wide clearance. On roads designed to carry two lanes of traffic, the proper widths for medium large structures are at least the width of slab needed to carry the traffic plus 8 ft. for primary roads and 4 ft. for secondary roads.

Advertisements and Other Obstructions

Any kind of a billboard or obstruction on the highway, which will detract from the driver's attention, is a contributing factor to accidents. For this reason, as well as for the sake of appearance, all advertisements should be eliminated from the right of way. In most States on the main highways this has been done; in some places, however, they have re-appeared on warning signs. Perhaps all the advantages of the sign are sacrificed by such practice. The habit of painting advertisements on the pavement should be stopped by imposing a severe penalty. Likewise, the method of erecting crosses along the highway where fatalities have occurred is of doubtful value.

Warning Signs

During recent years traffic has increased rapidly, out of proportion to the limited finances for coping with it. Hence, nearly every highway system in America has many dangerous places, some of which cannot be eliminated for years to come. The only recourse is to warn the traveler. Flashlight signals

should be installed at practically every grade crossing of an important highway with a high-speed railroad. In Indiana there are 511 grade railroad crossings on the State Highway System and only about 100 of these are protected by watchmen, gates, or suitable flash signals. Of all fatalities reported on this System, 36% in 1925 and 25% in 1926 occurred at railroad grade crossings (Ohio records show about 30 per cent.). It seems impossible to reduce these fatalities without giving greater protection to the public at such places.

The State should protect the unsuspecting driver against all dangerous places in the highway with ample markers. A standard system for this purpose adopted by the American Association of State Officials seems to work satisfactorily. It should be adopted universally so that the traveler will not be confused by different systems in different States.

Safety compaigns through newspaper publicity have been waged throughout the nation to influence the motorist and thus bring about saner driving. Many people have felt discouraged because of the constantly increasing number of accidents and have concluded that these compaigns were of little value. The writer, on the contrary, believes the most valuable means of increasing safety on highways is to continue such campaigns. Because accidents have not decreased it does not follow that such campaigns have not been beneficial. The greater speeds of travel together with the increased number of vehicles are responsible.

The engineer's duty in the reduction of accidents is a very important one. Although the source of danger preventable by his work is limited, perhaps it can be more easily controlled than any other. The improvements in the automobile bring increases in the speed; the engineer must cope with this problem. Much can yet be done. Little, however, can be accomplished without study and effort. It is far better to recognize this and put forth every effort to increase the safety. The opposite attitude once was taken by a prominent official when he became disgusted with the high-speed traffic and "gave up" with the remark, "Let the damned fools kill themselves". Nothing is so hopeful as making an effort and this is the responsibility and duty to the general public owed by engineers.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION ON "A NATIONAL RECLAMATION POLICY"

This report, scheduled for discussion at a Division Meeting (San Diego, Calif., October 4, 1928), is printed in advance as information in order to furnish opportunity for the presentation of written or oral comments.

Historical Statement

Under date of November 1, 1926, Joseph Jacobs, Chairman of the Executive Committee of the Irrigation Division of the Society, wrote as follows:

"The Executive Committee of the Irrigation Division at its last meeting authorized the appointment of a new committee to be designated 'Committee on A National Reclamation Policy'. In accordance with the thought that the Committee should be fairly large, should be well distributed geographically, and should be comprised of men of long experience in irrigation matters, its personnel was made up as follows:

"J. B. Lippincott, Consulting Hydraulic Engineer, Los Angeles, Calif., *Chairman*.
G. M. Bacon, State Engineer, Salt Lake City, Utah.
C. E. Grunsky, Consulting Engineer, San Francisco, Calif.
M. C. Hinderlider, State Engineer, Denver, Colo.
W. H. Code, Consulting Engineer, Los Angeles, Calif.
Burton Smith, General Manager, Twin Falls Canal Company, Twin Falls, Idaho.
G. E. P. Smith, University of Arizona, Tucson, Ariz.
J. C. Stevens, Consulting Engineer, Portland, Ore.
R. K. Tiffany, State Hydraulic Engineer, Olympia, Wash.
John E. Field, Consulting Engineer, Denver, Colo.
Joseph A. Elliott, Manager and Vice-President, Wyoming Development Company, Wheatland, Wyo."

An effort was made to exchange views between these members by correspondence during the past year, but it proved unsatisfactory. A meeting of this Committee was called for the session of the Society in Denver, Colo., in July, 1927, but a quorum was not present and no definite action was taken. The Committee, however, held a session in Los Angeles, Calif., on September 19 to 21, inclusive, 1927, at which were present: Messrs. G. M. Bacon, J. C.

Stevens, John E. Field, W. H. Code, C. E. Grunsky, and J. B. Lippincott. As this constituted a quorum the Committee proceeded to business and adopted a "Progress Report" under date of September 21, 1927.

It was decided that supporting statements for this report should be written by the different members of the Committee to whom separate paragraphs were assigned and that an Editing Committee composed of Messrs. Grunsky, Code, and Lippincott should meet in San Francisco, Calif., about October 10 to review them.

Following the meeting of September, 1927, Mr. Code wrote a Minority Report which he submitted to all the members of the Committee for their consideration. In addition, a supporting statement was prepared by the Committee covering the Majority Report. The vote by letter of the eleven members of the Committee on the adoption of the report was as follows: Seven in favor of the adoption of the Majority Report, three in favor of the adoption of Mr. Code's substitute report. Seven members of the Committee favored considering the Majority Report of September, 1927, as a progress report which should be given further consideration by the Committee.

In accordance with the decision of the Committee to consider the report of September, 1927, as a progress report, a second meeting of the Committee was held in San Francisco on July 2 and 3, 1928. At that meeting there was present the following members: Messrs. C. E. Grunsky, W. H. Code, G. M. Bacon, G. E. P. Smith, J. C. Stevens, John E. Field, and J. B. Lippincott. This Committee reviewed both reports and the supporting statements and those present unanimously adopted a report dated July 3, 1928, which is a modification in some respects of the report of September, 1927. Copies of the modified report were sent to the members of the Committee not in attendance at this meeting.

Report

By resolution the Committee limited its work to a study of policies governing the reclamation of arid lands and related matters.

The policy of the United States and of the several States in the matter of water conservation and arid land reclamation should be controlled by certain basic principles as follows:

(1) The waiving of interest payments to land owners on Government reclamation projects is unwise. In the future Government contributions should appear in the assumption of a part of the cost of project works and not in the granting of relief to the individual farmer such as the waiving of interest charges.

The United States Bureau of Reclamation has formulated a program of construction, covering the ensuing ten years, involving expenditures of approximately $100 000 000. To the extent that commitments have been made the Bureau should fulfill its assumed obligations and, on the other hand, the land owner should be required to meet his obligations or surrender his holding in the Government project.

(2) The conservation of the water in the rivers and lakes of the country should be under public control; and in order to lay a proper foundation for the making of comprehensive plans the Federal and State Governments should gather data, compile statistics, and conduct studies necessary to determine the feasibility of projects.

(3) The regulation of the flow of streams for the prevention of floods and for the best possible utilization of the waters should be undertaken by the States, or jointly by the United States and the States under such suitable forms of co-operation as may be appropriate under the Constitutional authority now delegated to each. They should prepare and adopt comprehensive plans for such regulation and should bear an equitable portion of the cost of water storage and flood-control work when the economic aspects after full investigations are found to be favorable, and the remainder of the cost should be allocated to flood control, irrigation, power development, municipal water supply, and other purposes.

(4) Where protection against flood waters results from the regulation of stream flow by means of reservoirs or otherwise, the proportion of the cost of the flood-control work not assumed by the Federal or State Government should be assessed against the lands and other properties which receive benefit therefrom.

(5) Municipalities or other public agencies or private parties should be allowed to construct approved projects in conformity with the approved plans; subject, however, to the public control of reservoirs and to the recapture after a reasonable time by the public of any franchise or similar rights conferred on private parties.

(6) The output of power and of water at Federal or State works should be disposed of at wholesale and not at retail.

(7) Interested States under suitable interstate compacts should be permitted by the United States to undertake the regulation of interstate streams.

(8) In the carrying out of further stream-regulation work, preference should be given to the construction of regulating reservoirs and the development of supplemental water supplies for existing irrigation systems, whether Federal or otherwise.

(9) Agricultural conditions due to over-production are such at present that it is undesirable for the Federal Government, except in the case of commitments already made, to bring new areas under cultivation.

(10) The construction of new irrigation projects should not be authorized except after thorough investigation and favorable recommendation by a Board of Review. This Board should include competent construction engineers, engineers with special operating and agricultural experience, economists, and financiers familiar with local production and marketing conditions. The State should share in the responsibility for the selection and approval of projects. In determining the feasibility of proposed projects, State lines, local interests, and political expediency should not control.

(11) When new projects are authorized, principal and interest payments on construction costs should be required of the land owner. The interest

rate should be low and the principal payments extended over a long period, with no payments on principal during the early years.

(12) The plan of repayment of construction costs of reclamation should be put into operation on each unit of the project at an early date after completion. The plan of payment should be sufficiently elastic to meet the settlers' ability to pay, but no relinquishment in the terms of repayment, once they are put into effect, should be permitted.

(13) In the case of reclamation projects, it should be recognized that settlement of the land is fully as imperative to success as construction. It can be greatly stimulated by the Government or other authorities taking drastic measures to prevent land speculation. The Department of the Interior is to be congratulated on its efforts to curb land speculation in recently authorized projects.

(14) Land settlement including paternalistic, financial, or any other kind of assistance to the individual farmer should be treated as a local matter and should therefore be made the concern of the State or locality rather than of the United States. Aid extended by Federal Land Banks has been generally helpful and the possibility of further extension of such aid by this or similar agencies is worthy of serious consideration.

(15) The Federal Government should continue its present policy of relinquishing control of completed works to suitably organized local agencies as soon as practicable.

(16) According to the report of the U. S. Bureau of Reclamation for 1926, the total area of land provided with water for irrigation was 1 803 000 acres in reclamation projects, of which, 1 320 000 acres were being cultivated by irrigation—a record of which the country may well be proud.

J. B. LIPPINCOTT, *Chairman,*
G. M. BACON,
W. H. CODE,
J. A. ELLIOTT,
JOHN E. FIELD,
C. E. GRUNSKY,
M. C. HINDERLIDER,
BURTON SMITH,
G. E. P. SMITH,
J. C. STEVENS,
R. K. TIFFANY,
Committee.

July 3, 1928.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

ADVANCES IN WATERWAYS ENGINEERING DURING A HALF CENTURY

Discussion*

By W. M. Black, M. Am. Soc. C. E.†

W. M. Black,‡ M. Am. Soc. C. E. (by letter).§—Major Hall‖ brings out two points which were omitted from the paper. The type of dredge described as developed on the Mississippi River is efficient only for handling the very finely divided materials which form the bars in that river below the mouth of the Missouri, as well as in the Missouri, Arkansas, and other of the tributaries of the lower river. In the upper sections of the Mississippi River, above the mouth of the Missouri, the materials forming the bed of the stream are coarser and for their removal require the application of a greater cutting power than that afforded by the water jets of the lower-river type of dredge. The "sand dams" of the upper river mark an advance in the art of river improvement which should have been noted.

The comments of Mr. Barlow¶ are also of value. To avoid misconceptions some further discussion of the costs of dredging seems desirable. Mr. Barlow is in error when he states:¶

"* * * that the total yardage on which the Government figures of approximately 8 cents are based, is about 1 500 000 per year, a very trifling amount; whereas the contract yardages, from which the average prices are obtained, vary between 13 800 000 and 25 000 000 per year".

A more careful scrutiny of Table 2** will show that the figures for "average performances" give the average of the work of 42 dredges in 1923 and of 43 dredges in 1924 and 1925, operating in approximately 20 different districts and having a total output varying between 61 000 000 and 72 000 000 cu. yd. per year.

* Discussion of the paper by W. M. Black, M. Am. Soc. C. E., continued from February, 1928, *Proceedings*.

† Author's closure.

‡ Maj. Gen., U. S. A. (*Retired*); Engr. (Black, McKenney & Stewart), Washington, D. C.

§ Received by the Secretary, April 26, 1928.

‖ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 643.

¶ *Loc. cit.*, p. 644.

** *Loc. cit.*, October, 1927, Papers and Discussions, p. 1950.

With respect to the over-depth dredging, each contract makes provision for payment for the removal of materials from below the contract depth, usually for 1 ft. or 2 ft., in order to make due allowance for the unavoidable irregularity of the bottom of a cut. The statement* that 1 500 000 or 2 000 000 cu. yd. "very often" must be removed in order to be paid for 1 000 000 cu. yd. of dredged material, may be true for exceptional cases. However, available records show that, as a rule the yardage of "unpaid-for" over-depth dredging varies between zero and 30% of the yardage paid for. In certain cases dredging beyond over-depth limits is credited to the output of Government dredges, but the amount of such yardage is very small. Any extensive over-depth dredging by Government plant is done intentionally to avoid the necessity for early maintenance dredging.

The statement as to the method of computing the yardage excavated by dredges is in error.* As a rule the yardage is computed from a comparison of surveys made before and after the dredging. These surveys are made by a field party, not connected with the dredging operation and reporting direct to the District Office. The same procedure is followed in measuring yardage in contract dredging.

The cost figures for the Government work include all operating expenses except plant rental and depreciation, interest on investment, and fire, marine, and employers' liability insurance. The cost of the repairs and of plant maintenance is absorbed in the cost of the work.

The record for eleven years (1916 to 1926, inclusive) shows that the yearly average of losses to Government floating plant by fire, by collisions, and from miscellaneous causes is $164 195.47. The estimated value of the plant in 1925 was $47 654 706.00. The losses thus averaged $0.003446 per year per dollar of the investment.

The cost figures given for contract work in Table 3† do not show the entire cost of the work to the United States since they do not include the costs of surveys, of inspection, or of office overhead.

The charges for the rental of Government plant are made high purposely in order that such plant shall not be used by private individuals in competition with a contractor's plant. The illustration given by Mr. Barlow of the handicap formed by rental charges is misleading in that there is no Government pipe-line dredging unit which has cost $1 000 000. In very few cases is the valuation as high as $500 000.

It is the policy of the Engineer Department to have dredging done by contract unless there is too great a difference between the cost of contract work and the cost of work with Government plant. In order to be able to judge relative costs more clearly, since 1926 the cost accounting system has included rental and depreciation on all items of floating plant. In estimating the costs of dredging with Government plant in comparison with bids by contractors the Department takes into consideration interest on investment (on value of plant used), and fire, marine, and employers' liability insurance. These, however, are not included in the published cost figures as actual charges against the work.

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 644.

† *Loc. cit.*, October, 1927, Papers and Discussions, p. 1951.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE HEAD-WORKS OF THE IMPERIAL CANAL

Discussion*

By E. S. Lindley, M. Am. Soc. C. E.

E. S. Lindley,† M. Am. Soc. C. E. (by letter).‡—The account of the work given in this paper shows practice that differs in several respects from that followed in India. It seems worth while to describe the latter for comparison; but there is no intention of implying, without any knowledge of local conditions, that it would have been applicable in California.

The writer wishes to discuss two points namely, silt exclusion and building on a foundation of sand. The 500 000 cu. yd. per month reported§ removed by dredges are equivalent to 3 620 acre-ft. per year. Comparing that figure with the earlier estimate of 12 000 acre-ft. per year of silt admitted, and the later estimate of 14 000 acre-ft., with a maximum of 24 900 acre-ft. in one year, it is clear that dredging cannot compete with measures of silt exclusion. Even if as much silt were dredged in a year as was admitted, trouble would still be experienced at times. Under Panjab conditions of river flow, probably about three-quarters of the total silt of the year passes in the four worst months (a flood season when Himalayan snows melt and are supplemented by monsoon rainfall).

The new Rockwood Gate was designed "so that mainly surface water would enter the canal". The design seems to have followed the common tacit assumption that provision of a high sill excludes from the regulator any water which flows in the parent channel lower than the plane of the sill. If the matter is regarded from the point of view of the draw or suck exercised by the offtake at different points in the cross-section of the parent channel, it will be realized

* This discussion (of the paper by C. E. Grunsky, Past-President, Am. Soc. C. E., published in November, 1927, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Superintending Engr., Indian Public Works Dept., Panjab Irrig. Branch; Wotton-under-Edge, Gloucester, England.

‡ Received by the Secretary, April 17, 1928.

§ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2185.

that the cleavage between the water taken in and that passed on, is vertical and not horizontal; and that, except for the behavior of the heaviest particles in a vertical current, a high sill admits the same silt as an under-shot gate, because it does not change this cleavage.

Investigation is now showing that the cleavage is sensibly vertical over the greater part of the depth in the parent channel; but near the bed, where forward velocities are lowest and silt conditions are worst, flow to the offtake comes from a greater width, to a degree considerably more than would ordinarily be expected. A side offtake, therefore, must draw a large share of the more silty water to the exclusion of cleaner surface water.

The author refers* to local observations showing that the bottom water contains the most silt. If sampling could be carried close to the bed without disturbing the bed itself, the water would be found to be even dirtier than stated.

Observations at other places have shown, not only that the average silt charge is greater at the bottom than near the surface, but that this concentration is greater in the coarser grades of the silt carried. Heavy silt is absent from the surface water, and the heaviest is confined to the bottom water, unless the current is turbulent. Viewed in the light of these considerations, it can be understood that a side offtake is a selector of silt, or the opposite of a filter. In developing the design of offtakes intended for silt exclusion from this point of view, the object is to secure the utmost concentration of silt in parts of the approaching current which are then excluded. The first step involves conducting the flow of the parent channel for a sufficient distance in a smooth channel, at a velocity that just avoids depositing silt on the bed, and also avoids eddies and boils as much as possible.

For the second step, a device that is applicable in some cases, consists of smoothly curved vanes on the bed, which guide the dirty water away from the offtake, give the whole current a twist, and thus also bring it clean water on the surface. This device first suggested by Mr. H. W. King, of the Panjab Irrigation Department,† is thus the opposite to that used by Benjamin F. Groat, M. Am. Soc. C. E., to exclude ice.‡

Another suggestion, applicable on a large scale, is due to the late Mr. F. V. Elsden, of the Panjab Irrigation Department. For a new work he would build a regulator in two stories across the parent channel, in the position of the under-sluices of a Panjab head-works. The gates of the lower story would escape dirty bottom water down the river, while those of the upper story would take the cleaner upper flow to be conducted away in the canal.§

A plain high and long sill also has some effect in excluding the coarsest silt, without affecting the plane of cleavage of the parent channel. Any given grade of silt could approximately be excluded if the flow had nowhere an upward vertical velocity exceeding its rate of settlement in still water. This presupposes prevention of eddies to stir up the bottom silt or cause local

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2185.

† Panjab Eng. Congress, 1918 and 1920.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1138.

§ Panjab Irrig. Branch, *Paper No.* 25.

upward currents, and a slight current along the face of the sill to prevent the excluded silt from banking up there until it all enters again.

Referring to the second point for discussion, when a work is built on a pervious foundation to head up water, and without a cut-off to an impervious stratum, the difference of head causes flow through all the pervious medium, and there is a regular drop of head along any stream line. *Prima facie,* it might be expected that the easiest path (along which the greatest flow occurs) would be the shortest; that is, the envelope embracing the work and cutting off any re-entrant angles. However, observation of static pressures under such works has shown that distances in which equal falls of pressure occur should be measured along the "line of creep", or junction between the work and foundation, along both vertical and horizontal lines. The weight of the work must exceed the total upward pressures indicated by the line of hydraulic gradient thus drawn. According as to whether or not the work is tied together by reinforcement, the weight must be suitably distributed for stability, or must be sufficient from point to point.

The shorter the line of creep the steeper the gradient and the greater the velocity of flow. For any given material there is a limiting velocity which must not be exceeded to avoid sand being blown at the toe of the work, with consequent piping. In that material such a velocity will be caused by a certain gradient, which is the (maximum) limiting gradient for that material. Therefore, with a given heading up, this gradient gives the minimum length of line of creep for the work. To exceed that length would add unnecessarily to the dimensions of the work and the weight required to balance upward pressures.

The limiting gradient for the sandy beds of Panjab rivers about 50 miles from their emergence from the foot-hills is generally taken as 1 in 15. In the finer loam of the plains between rivers, a gradient of 1 in 4 is used, but for finer pure sand it is flatter than 1 in 15. The length of the line of creep under the Rockwood Gate is about 80 ft. With the canal empty, the maximum head would be nearly 11½ ft., which would give a gradient of 1 in 7. The gradient for the Chowchilla regulator would seem to be about 1 in 6. As the sand is described as very fine and free of clay, these works seem very short, on these data and judged by this criterion. With a given length of line of creep, if the gates are moved down stream a greater part of the work has water at full depth standing on it, leaving a smaller part of the upward pressures to be balanced by the weight of the work. Therefore, it is economical to provide an impervious up-stream apron, especially as it is generally desirable to protect the bed against scour, and to have a smooth floor, thereby causing less eddies and raising of silt in the stream.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

MAXIMUM FLOOD DISCHARGE IN SAN JOAQUIN VALLEY, CALIFORNIA

Discussion*

By Oren Reed, Assoc. M. Am. Soc. C. E.†

Oren Reed,‡ Assoc. M. Am. Soc. C. E. (by letter).§—A review of recent engineering literature would indicate that flood intensity and frequency have not been given their due importance. Widespread interest has been manifested after each major disaster, but more studies should be made for preventative purposes. The year 1927 was a remarkable period for great floods claiming the attention of the engineer. In them, he finds a need for thorough study as a means of coping with repetitions.

When floods occur, men instinctively feel that they are helpless against such occurrences, and that they must continue to bear them even in the future. It is recognized by engineers, of course, that there have always been floods and that their frequency and intensity have not changed greatly. Loss of property and life have steadily increased because Man has encroached on the river banks.|| When structures are erected on the bank of a river, engineers must design with reference to flood discharges and flood stages. Their only safe criteria in estimating floods of the future must be obtained from a study of floods of the past. At present, unfortunately, the knowledge of flood facts is quite inadequate.

The record of flood run-off from any California river is too short to estimate accurately its probable maximum flood. Since 1918 the precipitation has been sub-normal, and no major floods have occurred. The passage of years will make the record more complete, but in the meantime existing records must be used.

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* Discussion of the paper by Oren Reed, Assoc. M. Am. Soc. C. E., continued from April, 1928, *Proceedings*.

† Author's closure.

‡ Asst. Designing Engr., San Joaquin Light & Power Corp., Fresno, Calif.

§ Received by the Secretary, March 18, 1928.

|| *Engineering News-Record*, November 10, 1927, p. 741.

Further Flood Study.—As stated by Mr. Fox,* the writer's methods of studying flood records have been changed in certain details. In Central California, as in other regions, the geographic location of an area in respect to the path of cyclonic storms determines to a great extent its flood run-off. The primary data are used to determine only one curve, such as Fig. 7,† which was called the "geographical location curve".

The relation of average elevations and size of drainage areas is, however, of major importance. Storms which produce the maximum flood run-off on the Kings River occur in January and February. At this season precipitation on the upper part of the water-shed is in the form of snow and contributes only a small run-off. Due to this fact the productive or effective area would be a relatively small part of the total. This extended study has been based on effective areas.

Temperature data are not published for many mountain stations in California. Reliable records are available for the Fresno-Auberry-Huntington Lake Section, on the San Joaquin River, and for the Sacramento-Auburn-Colfax-Norden Section, on the American River. (See Fig. 19.) The American River was not used for the run-off records, but may be used in the study of temperature variation without error. The American River is about 35 miles north of the Cosumnes River, the most northerly stream in the run-off study.

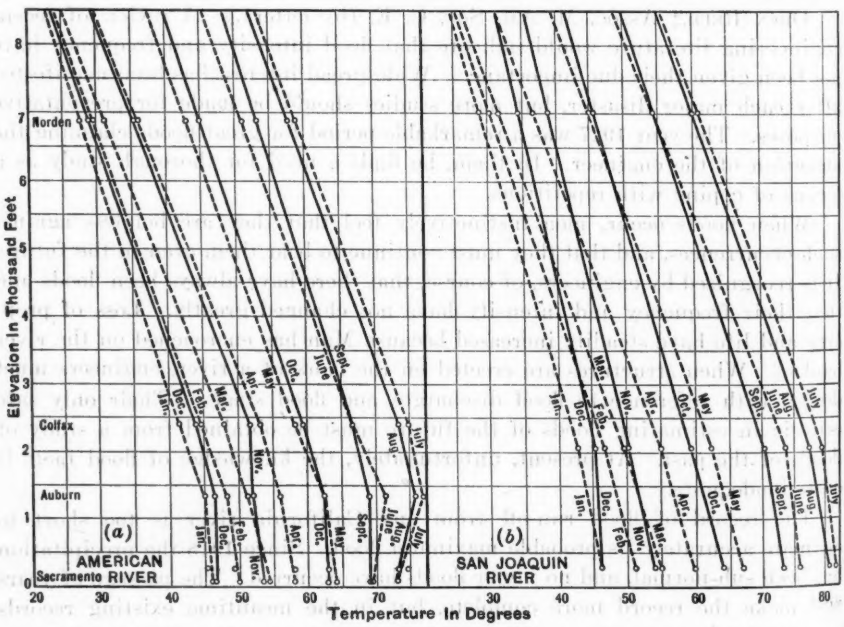

FIG. 19.—MEAN MONTHLY TEMPERATURES, AMERICAN AND SAN JOAQUIN RIVERS.

A study of meteorological data shows that the mean daily temperature during a storm period is from 4 to 8° higher than the mean monthly tempera-

* *Proceedings,* Am. Soc. C. E., February, 1928, Papers and Discussions, p. 652.

† *Loc. cit.,* November, 1927, Papers and Discussions, p. 2234.

ture. Based on that fact, the assumption has been made for this study that the run-off would be so slight that it could be neglected when the mean monthly temperature was below 22.5° Fahr. Fig. 20 has been drawn from the data given on Fig. 19 to show the relation of altitude to mean January and February temperature of 22.5° for the eleven streams. A curve for 25° is also given for reference. The curves are based on observations at only two stations and are plotted as straight lines. No doubt, there would be some variation from a straight line, but the deviation should not be great. This statement is substantiated by the temperature observations at the Upper Kings River meteorological stations. The Kings River stations have been maintained for six years.

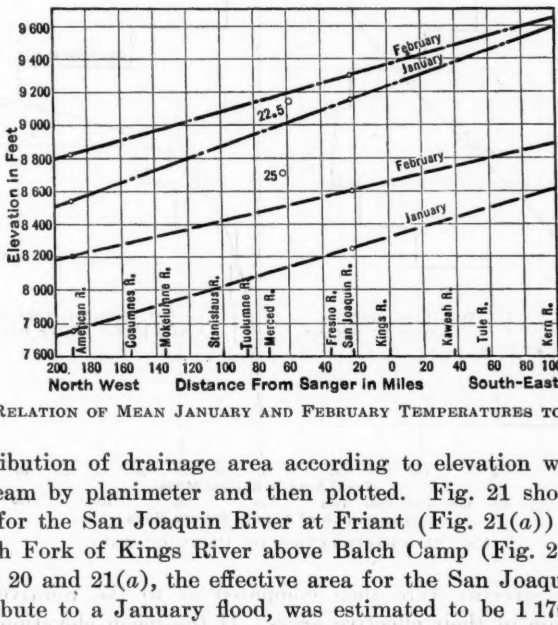

FIG. 20.—RELATION OF MEAN JANUARY AND FEBRUARY TEMPERATURES TO ELEVATION.

The distribution of drainage area according to elevation was determined for each stream by planimeter and then plotted. Fig. 21 shows two of the curves; one for the San Joaquin River at Friant (Fig. 21(*a*)) and the other for the North Fork of Kings River above Balch Camp (Fig. 21(*b*)). Referring to Figs. 20 and 21(*a*), the effective area for the San Joaquin River, that would contribute to a January flood, was estimated to be 1 170 sq. miles, or 71.4% of the total area. The average elevation of the effective area was 5 630 ft. For the North Fork of Kings River above Balch Camp the effective area for a January flood is 159.6 sq. miles and the average elevation, 7 490 ft. In like manner, the effective area and its average elevation were determined for each stream in turn.

It will be noted from Fig. 3,* that the greatest precipitation from a normal storm on Kings River is about 6 000 ft. The locus of condensation on a mountain slope is usually at a lower level in winter than in summer, but this is offset to a certain extent by the low run-off of the upper part of the contributing area. Sufficient precipitation data were not available for each stream for plotting curves similar to Fig. 3 and only five curves were used, the Kern, Kings, San Joaquin, Mokelumne, and American Rivers. The San Joaquin River Section is indicated on Fig. 22. Each curve was used for two or three

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2225.

adjoining streams; the American River curve was used for the Cosumnes River only. In plotting these curves, the mean elevation of the slopes back of a meteorological station was plotted rather than the actual elevation of the station. Most meteorological stations in the Sierras are in deep river canyons while the precipitation corresponds to the elevation of the slopes immediately to the leeward of these stations. Considerable error may be introduced when actual elevations of the stations are used.

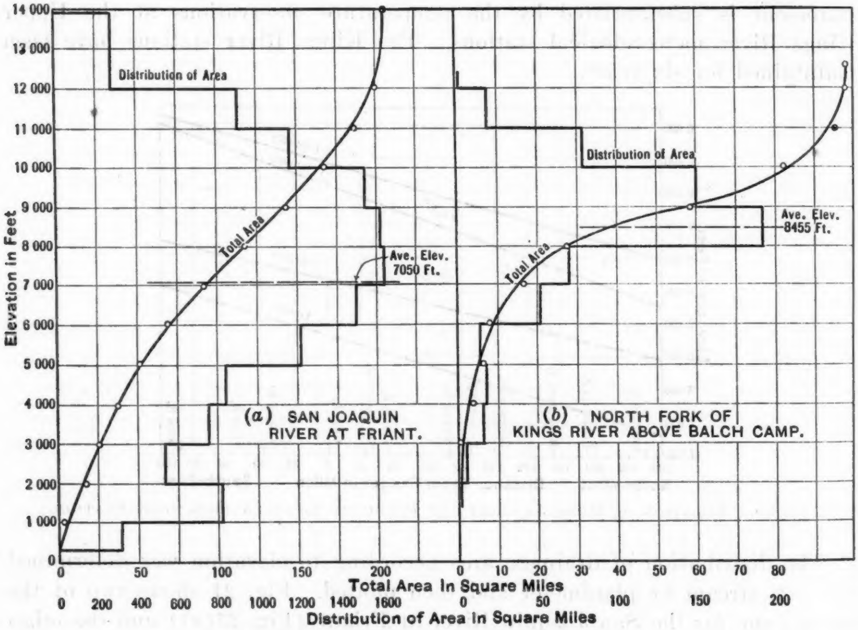

Fig. 21.—Distribution of Drainage Area.

The eleven streams were then compared as to the relative location of average elevation of their effective areas. If the mean elevation of a stream coincided with the point of maximum precipitation, that water-shed would be likely to receive heavy precipitation from each general storm and, therefore, it was given a factor of 1.0. If the average elevation was greater or less than the elevation of maximum precipitation, the factor was suitably lowered. In this manner, factors were determined for evaluating the effect of average elevations of the eleven streams.

Most flood formulas have been based on the variation of flood run-off with size of drainage area. The variation of flood concentration and run-off with area has been ably presented by C. S. Jarvis, M. Am. Soc. C. E.* In order to decrease the effect of difference in precipitation from north to south in the Sierra Nevada Mountains (Fig. 2†), only four major streams were used in determining the curve in Fig. 23—the Kings, Kaweah, San Joaquin, and

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 985.

† *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2224.

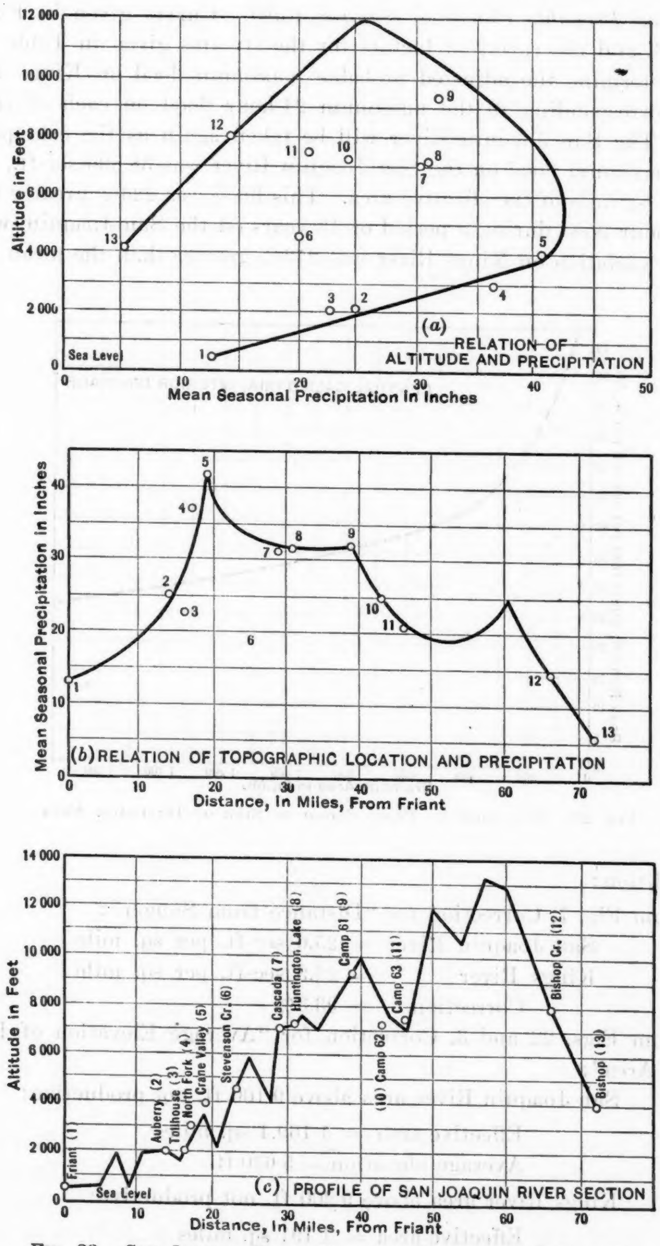

FIG. 22.—SAN JOAQUIN RIVER GROUP OF PRECIPITATION GAUGES.

Merced Rivers. Records for these primary streams and several of their tributaries were plotted.

Adjusted Probable Floods at Sanger, Calif.—Curves given in Figs. 7, 3, 22, and 23, and the variation factors for the streams given in Table 1* were used to determine the adjusted probable maximum flood on Kings River at Sanger, corresponding to the maximum 24-hour flood on each of the other streams. The San Joaquin River will be taken again as the example. The 24-hour maximum flood on the San Joaquin River was 38 800 sec-ft., or 33.16 sec-ft. per sq. mile of the effective area. This flood was 226% greater than the mean 24-hour flood during a period of 13 years on the San Joaquin, while the maximum variation on Kings River was 253% greater than the mean.

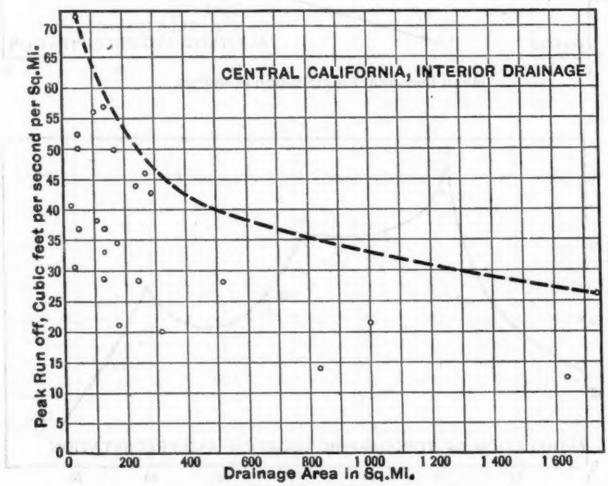

FIG. 23.—RELATION OF PEAK FLOOD TO SIZE OF DRAINAGE AREA.

In addition:

From Fig. 7, Correction for "Distance from Sanger":

San Joaquin River	=	25.6 sec-ft. per sq. mile
Kings River	=	23.9 sec-ft. per sq. mile
Correction	=	93.3%

From Figs. 22 and 3, Correction for "Average Elevation of Drainage Area":

San Joaquin River area above 9 100 ft. not productive:

Effective area = 1 169.4 sq. miles
Average elevation = 5 630 ft.

Kings River area above 9 200 ft. not productive:

Effective area = 1 191 sq. miles
Average elevation = 5 230 ft.

* *Proceedings*, Am. Soc. C. E., November, 1927, p. 2229.

San Joaquin River:

$$\text{Relative wetness} = \frac{43}{43} = 1.00$$

Kings River:

$$\text{Relative wetness} = \frac{46}{47.2} = 0.977$$

$$\text{Correction} = \frac{0.977}{1.00} = 97.7\%$$

From Fig 23, Correction for "Drainage Area in Square Miles":

San Joaquin River... 1 169.4 sq. miles = 31.5 sec-ft. per sq. mile
Kings River.......... 1 191 sq. miles = 31.4 sec-ft. per sq. mile
Correction.................. = 99.7%

The maximum probable 24-hour flood at Sanger is,

$$33.16 \times \frac{253}{226} \times 93.3 \times 97.7 \times 99.7 = 33.74 \text{ sec-ft. per sq. mile}$$

or, 40 200 sec-ft. from the effective area.

From Fig. 10,* a peak flood of 70 300 sec-ft., or 59.0 sec-ft. per sq. mile of contributing area, would be possible from a 24-hour flood of 40 200 sec-ft.

The other ten streams were studied in like manner and gave results as shown in Table 11.

TABLE 11.—DEDUCED FLOOD FLOW FOR KINGS RIVER NEAR SANGER, CALIFORNIA.

Stream.	24-hour flood, in second-feet per square mile, effective area.	Corresponding 24-hour flood at Sanger, in second-feet per square mile, effective area.	PEAK FLOOD, KINGS RIVER, AT SANGER.		
			Second-feet per square mile.		Second-feet.
			Effective area.	Total area.	
Kern	8.08	17.09	43.1	29.5	51 300
Tule	20.55	24.10	48.2	33.0	57 400
Kaweah	21.8	23.20	48.2	33.0	57 400
Kings	36.9	36.9	61.2	41.9	73 000
San Joaquin	33.16	33.74	59.0	40.4	70 300
Fresno	28.07	23.3	47.7	32.6	56 800
Merced	42.2	26.6	51.5	35.3	61 400
Tuolumne	42.6	31.6	57.0	39.0	67 800
Stanislaus	42.9	26.6	51.8	35.5	61 800
Mokelumne	27.5	20.9	45.7	31.3	54 500
Cosumnès	42.67	25.5	49.9	34.2	59 500
Average	....	.....	51.2	35.1	61 000

The relation of the adjusted floods appears to be more reasonable than the values given in Table 3.† The average value of the peak flood for Kings River, at Sanger, is only 2% greater than the peak flood of 1914. Peak flood intensity, as in 1914, is estimated from an extension of the rating curve and may be somewhat inaccurate.

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2235.

† *Loc. cit.*, p. 2237.

Adjusted Probable Floods at Balch Camp, California.—The maximum peak flood on the North Fork at Balch Camp was determined by a similar method. The maximum 24-hour flood on the San Joaquin adjusted to suit Kings River conditions produced a peak flood of 59.0 sec-ft. per sq. mile of effective area at Sanger. The corresponding peak at Balch Camp was found to be:

$$59.0 \times 181.5 \times 100 = 107.1 \text{ sec-ft. per sq. mile of effective area}$$
$$= 17\,100 \text{ sec-ft.}$$

The results of the complete study are given in Table 12.

TABLE 12.—DEDUCED FLOOD FLOW FOR NORTH FORK OF KINGS RIVER AT BALCH CAMP, CALIFORNIA.

Flood adjusted from	Peak flood on Kings River, at Sanger, in second-feet per square mile of effective area.	Peak Flood, North Fork of Kings River at Balch Camp.	
		In second-feet per square mile of effective area.	In second-feet.
Kern	43.1	78.2	12 500
Tule	48.2	87.4	13 950
Kaweah	48.2	87.4	13 950
Kings River	61.2	111.0	17 700
San Joaquin	59.0	107.1	17 100
Fresno	47.7	86.5	13 800
Merced	51.5	93.5	14 920
Tuolumne	57.0	105.2	16 800
Stanislaus	51.8	94.2	15 030
Mokelumne	45.7	82.9	13 240
Cosumnes	49.9	90.8	14 500
Average	51.2	93.1	14 860

Effect of Storage Regulation.—Construction of a storage reservoir, Lake Wishon, is planned for a site near Cliff Camp, on the North Fork, at Elevation 6 550. When this dam is built, only that area between Lake Wishon and Balch Camp will contribute to a winter flood at Balch Camp, except in an extraordinary season. Usually the storage reservoir would be drawn down by the beginning of winter and all of a winter flood could be stored. The drainage area between Lake Wishon and Balch Camp is 71.27 sq. miles and its average elevation is 6 450 ft. The peak flood at Balch Camp from this intermediate area, estimated by the method outlined for the total area, would be 7 990 sec-ft., or 112.2 sec-ft. per sq. mile of effective area.

However, in a very wet series of years, the reservoir might be practically full at the time of a late winter storm. Pondage above the spillway level would reduce the flood peak, but the run-off from the drainage area above the storage dam would materially add to a flood at Balch Camp.

Flow for the Balch Tunnel is diverted from a small pond, which has a total capacity of 1 565 acre-ft. and a surface area of 37.75 acres at its maximum elevation. This pond will usually be kept full, and its effect on floods at Balch Camp will be negligible.

European Flood Experience.—Within the last few years great loss of life and property has been caused in various countries of Europe by storms of high intensity. In several cases the flood flow was greater than had been known possibly for centuries. In contrast to conditions in America where stream-flow records are limited to relatively short periods, most European countries have an unbroken record of river behavior for many centuries past. Recent experience there indicates that long-time contingencies have to be reckoned with in providing protection against floods.

The following empirical data, Table 13, which are based on many years of past experience, are applicable to streams of Germany and Central Europe.*

TABLE 13.—PRECIPITATION AND RUN-OFF FOR GERMAN STREAMS.

Location.	Precipitation, in inches per year.	Run-Off, in Second-Feet per Square Mile.	
		Average flow.	Flood flow.
Plains	23.6 to 27.6	0.53 to 0.68	4.08 to 6.80
Foot-hills	35.4 to 78.7	1.63 to 2.72	136.0 to 407.0
Mountains	70.8 to 86.6	2.04 to 4.76	54.3 to 136.0

Discussions.—Mr. Follansbee† gives some very interesting precipitation data for stations near the Continental Divide in Colorado. The writer thoroughly agrees that the topography to the windward side of the region being studied is a controlling factor in determining variation in precipitation. General storms which produce heavy precipitation on the west slope of the Sierra Nevada Mountains travel in a southeasterly course from the Pacific Ocean. Before reaching the mountain slopes the storms pass over the Central Valley for a distance of 150 to 250 miles. The valley floor is relatively flat and ranges in elevation from sea level to about 400 ft. at Bakersfield, in the south end of the area.

As R. E. Horton, M. Am. Soc. C. E., has pointed out,‡ the winds approaching the Continental Divide from the west and southwest must blow for hundreds of miles over the Rocky Mountain plateau, for which the dominant elevation is about 7 000 ft. These winds lose nearly all the moisture derived from the ocean, which they originally contained, in ascending to this plateau, and the precipitation which they yield thereafter is apparently derived mainly from moisture picked up en route from evaporation. The general plateau extends at a more or less uniform elevation of about 7 000 ft., nearly to the Continental Divide, where a rise of 4 000 to 6 000 ft. occurs within a distance of a comparatively few miles.

The temperature and relative humidity of these winds govern the amount of precipitation. The records indicate a moderate increase of precipitation with elevation up to a certain height, after which the increase is more rapid

* Hütte, Bd. III, 24th Edition, p. 634.

† *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 651.

‡ *Engineering News-Record*, February 28, 1924, p. 355.

to a maximum close to Elevation 11 000. The rate of increase at Corona on the Divide is less than at lower elevations. Pike's Peak is situated to the east of the Continental Divide and receives less precipitation than the corresponding elevation on the west slope. After the winds pass over the Continental Divide they are relatively dry, and precipitation at Pike's Peak is slight. If the distance between mountain chains is sufficient, the relative humidity of the winds will be increased, due to moisture derived from evaporation, as stated, and the second range of mountains will give rise to a second belt of high precipitation. This is illustrated by the heavy precipitation on the slopes of the Wasatch and Uinta Mountains in Utah. A case similar to that at Pike's Peak is shown by Fig. 22. The secondary ridge near Crane Valley receives high precipitation and robs the remainder of the San Joaquin drainage area of a considerable portion of its normal precipitation.

Many engineers have questioned the theory that precipitation decreases above a certain elevation. The contention is made that the records of precipitation for high elevations are insufficient in number and accuracy to warrant the general conclusion. It is true that the stations are few in number and that the accuracy of some of the records could be greatly improved. However, each case studied showed the same characteristics. Some of the stations, especially on the American River Section, have been established for many years and are well maintained.

The percentage of precipitation appearing as run-off increases up to an elevation of approximately 10 000 ft. in the Sierra Nevada Mountains (see Fig. 24). This is due primarily to a decrease in water losses (evaporation, transpiration, etc.) above the timber belt, which thins out above 7 000 ft. The relation of run-off to precipitation for German rivers is given in Table 14.*

TABLE 14.—PERCENTAGE OF PRECIPITATION APPEARING AS RUN-OFF FOR GERMAN RIVERS.

Location.	Ratio of run-off to precipitation, in percentage.
Plains	25 to 30
Foot-hills	50 to 70
Mountains	85

A similar relation, no doubt, holds for the area studied and a relatively high run-off per square mile would be expected from the higher drainage areas.

The points covered by Mr. Fox in his discussion have been previously enlarged upon by the writer. One fact might be further emphasized; that is, until many more records of precipitation and run-off are obtained, reliable estimates of probable future floods on any particular stream are difficult to make unless all the comparable data are used.

Mr. Lee's discussion† is a valuable contribution to a study of flood discharge. When sufficient data are available, the rational method is the most

* Hütte, Bd. III, 24th Edition, p. 634.

† *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1235.

reliable for studying flood behavior of a stream. Accurate records of precipitation intensity and amount must be collected for a considerable period for the area under consideration. The flood run-off factor can only be established by many observations. These data are largely lacking for drainage areas in the Sierra Nevada Mountains, and many years will be required to perfect the application of the rational method.

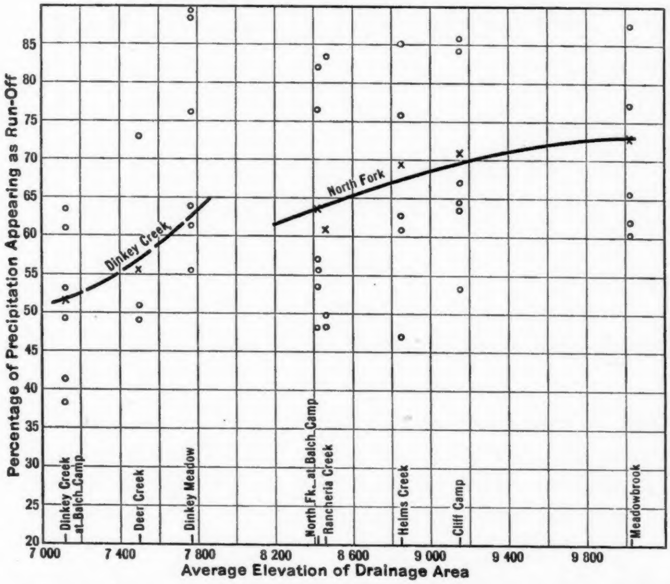

FIG. 24.—PERCENTAGE OF PRECIPITATION APPEARING AS RUN-OFF AT DIFFERENT ELEVATIONS, DINKEY CREEK AND NORTH FORK OF KINGS RIVER.

In the stream-group method, as used by the writer, the combination of intensity of rainfall and the percentage of resulting run-off make up the basic data. The record of maximum, 24-hour flood run-off for each primary stream, as determined by actual observation, was used in determining the equivalent record for the Kings River at Sanger.

In conclusion, the writer wishes to thank those who have contributed to the discussion of his paper and thus added materially to its value.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

UNUSUAL ENGINEERING FEATURES OF AN IMMENSE THEATRE BUILDING

Discussion*

By Messrs. A. Floris, Laurence J. Waller, and R. McC. Beanfield.†

A. Floris,‡ Esq. (by letter).§—The most interesting part of this structure is doubtless the Banquet Hall. Unfortunately, however, the author's statements with regard to this important part of the design are lacking in detail to such an extent that it is difficult to understand clearly the intentions of the designer.

The structure (as shown in Fig. 16‖), represents a rigid frame which, due to the absence of hinges or other design features eliminating bending moments, is statically indeterminate to a high degree. Obviously, to analyze such a complicated structure is practically not feasible and for this reason some simplifications must be introduced.

In such cases it is always possible, by a proper arrangement of hinges and other features of design, to divide the structure into secondary frames, beams, and columns the analysis of which is an easy matter. The structure (Fig. 18) may be divided into a two-hinged frame that supports the beams from the outside columns as well as the simple beam within the middle span. Also, it could be assumed to consist of two superimposed frames with two hinges each. The upper frame, with a tie-rod connecting two hinges, would be similar to that suggested first and the lower frame would be composed of the two columns and the straight beam within the middle span. Formulas for the solution of these frames are available elsewhere.¶

* Discussion of the paper by R. McC. Beanfield, Assoc. M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Author's closure.

‡ Los Angeles, Calif.

§ Received by the Secretary, April 18, 1928.

‖ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2667.

¶ "Rahmenformeln," by A. Kleinlogel, Berlin, 1925.

The solution of the latter subdivision results in reducing the bending moment of the beam in the middle of the lower frame. The same result can be obtained if this beam (Fig. 18) is fixed to the columns of the frame. In order to reduce the statically indeterminate quantities in such a case, a hinge is inserted in the top of the roof (Fig. 19) which simplifies the calculations appreciably.

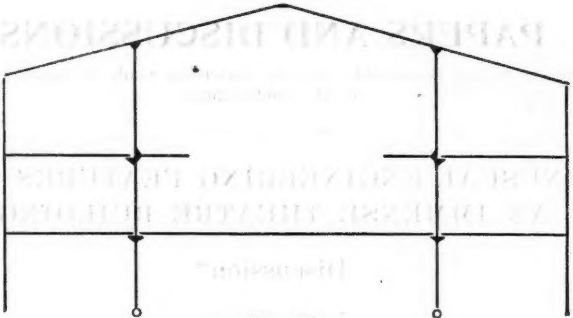

FIG. 18.

If the loading is symmetrical, the system thus formed is statically indeterminate in the second degree. This assumption as to the loading corresponds fairly well to the actual conditions with the exception, perhaps, of the wind pressure acting on the frame. In this building, however, the influence of wind seems to be negligible.

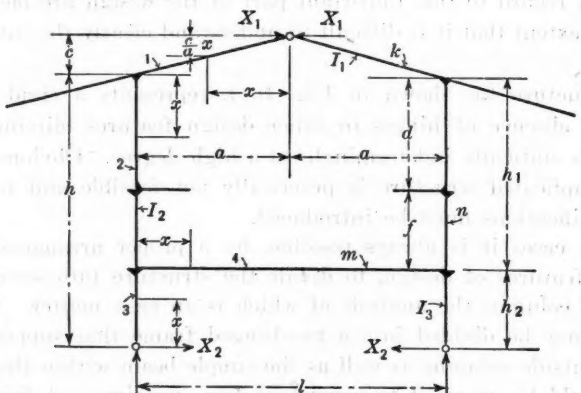

FIG. 19.

In the case of a symmetrical loading, the horizontal thrusts, X_1 and X_2 (Fig. 19), are taken as the unknown quantities. Then, if only the work done by the bending moments is considered, the equations of least work* are:

$$\frac{\delta u}{\delta X_1} = \int \frac{M}{E I} \frac{\delta M}{\delta X_1} d s = 0 \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$\frac{\delta u}{\delta X_2} = \int \frac{M}{E I} \frac{\delta M}{\delta X_2} d s = 0 \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

* "Die neueren Methoden der Festigkeitslehre und der Statik der Baukonstruktionen," H. Müller-Breslau, Leipzig, 1924, p. 96; see, also "Mechanics of Internal Work," I. P. Church, N. Y., 1910, p. 63.

Denoting by M_0 the bending moment of the statically determinate frame for a given loading, the following moments and their derivatives for the various elements of the frame are obtained (see Fig. 19):

Element 1:

$$M = M_0 - X_1 \frac{c}{a} x; \frac{\delta M}{\delta X_1} = -\frac{c}{a} x; \frac{\delta M}{\delta X_2} = 0$$

Element 2:

$$M = M_0 - X_1 (c + x); \frac{\delta M}{\delta X_1} = -(c + x); \frac{\delta M}{\delta X_2} = 0$$

Element 3:

$$M = M_0 - X_2 x; \frac{\delta M}{\delta X_1} = 0; \frac{\delta M}{\delta X_2} = -x$$

Element 4:

$$M = M_0 - X_1 (c + h_1) - X_2 h_2; \frac{\delta M}{\delta X_1} = -(c + h_1); \frac{\delta M}{\delta X_2} = -h_2$$

Using these expressions, Equations (1) and (2) take the final form:

$$X_1 \left[2 \left\{ k\, s \frac{c^2}{a^3} \int_0^a x^2\, d\, x + n \int_0^{h_1} (c + x)^2\, d\, x \right\} + m\, (c + h_1)^2 \int_0^l d\, x \right] + X_2\, m\, h_2\, (c + h_1) \int_0^l d\, x = \alpha$$

$$X_1\, m\, (c + h_1)\, h_2 \int_0^l d\, x + X_2 \left[2 \int_0^{h_2} x^2\, d\, x + m\, h_2^{\,2} \int_0^l d\, x = \Gamma \right.$$

or,

$$X_1 \left[2 \left\{ k\, s \frac{c^2}{3} + n\, c\, h_1 \left(c + h_1 + \frac{h_1^{\,2}}{3\, c} \right) \right\} + m\, (c + h_1)^2\, l \right] + X_2\, m\, h_2\, (c + h_1)\, l = \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and,

$$X_1\, m\, h_2\, (c + h_1)\, l + X_2\, h_2^{\,2} \left(\frac{2}{3} h_2 + m\, l \right) = \Gamma \ldots\ldots\ldots\ldots (4)$$

in which,

$$\alpha = 2 \left[k\, s \frac{c}{a^2} \int_0^a M_0\, x\, d\, x + n \int_0^{h_1} M_0\, (c + x)\, d\, x \right] + m\, (c + h_1) \int_0^l M_0\, d\, x$$

$$\Gamma = 2 \int_0^{h_2} M_0\, x\, d\, x + m\, h_2 \int_0^l M_0\, d\, x$$

and,

$$k = \frac{I_3}{I_1};\ n = \frac{I_3}{I_2};\ \text{and } m = \frac{I_3}{I_4}$$

The frame was made statically determinate by removing the roof hinge (without connecting the separated parts) and also putting the right column on rollers (Fig. 20).

The integrals in Equations (3) and (4) depend on the loading and must be determined in each case separately. The final results are (Fig. 20):

Loading (*a*).—For a unit load, p, uniformly distributed over the entire span of the roof (cantilevers excluded),

$$\alpha = \frac{p\,a^2}{4}\left[k\,s\,c + 2\left\{n\,h_1\,(2\,c + h_1) + m\,l\,(c + h_1)\right\}\right]$$

$$\Gamma = m\,h_2\,p\,a^2$$

Loading (*b*).—For a unit load, p, uniformly distributed over the entire span of the beam:

$$\alpha = m\,(c + h_1)\,\frac{p\,l^3}{12}$$

$$\Gamma = m\,h_2\,\frac{p\,l^3}{12}$$

Loading (*c*).—For a loading due to the cantilevers of the roof:

$$\alpha = -\,\mu_1\left[2\,n\,h_1\left(c + \frac{h_1}{2}\right) + m\,l\,(c + h_1)\right]$$

$$\Gamma = -\,\mu_2\,m\,h_2\,l$$

in which, μ_2 is the moment of these cantilevers over their supports.

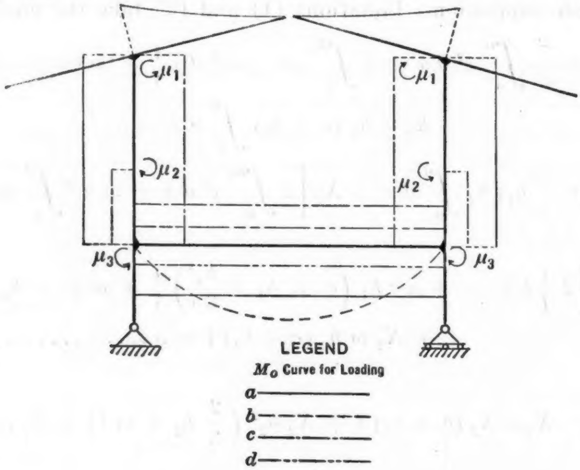

Fig. 20.

Loading (*d*).—For a loading due to the cantilevers or brackets above the elevation of the beam (for the inner brackets or cantilevers):

$$\alpha = \mu_2\left[2\,n\,f\left(c + d + \frac{f}{2}\right) + m\,l\,(c + h_1)\right]$$

$$\Gamma = \mu_2\,m\,h_2\,l$$

in which, μ_2 is the moment of the cantilevers over their supports. For the outer brackets the sign of μ_2 in the last two equations must be reversed.

Loading (*e*).—For a loading due to the brackets at the elevation of the beam:

$$\alpha = \mu_3\,m\,(c + h_1)\,l$$

$$\Gamma = \mu_3\,m\,h_2\,l$$

in which, μ_3 is the moment of these brackets at their supports.

If the influence of an unsymmetrical loading is to be considered, the rigid frame is statically indeterminate to the third degree. For the third unknown —the vertical component of the roof hinge—it is necessary that a third equation similar to Equations (1) and (2) be used.

For those who do not favor statically indeterminate structures, the frame represented in Fig. 18 can be made statically determinate by inserting a hinge in the top of the roof as is shown in Fig. 19. The result is a three-hinged frame, the analysis of which is well known.

LAURENCE J. WALLER,* ASSOC. M. AM. SOC. C. E. (by letter).†—The author has written a concise and yet complete description of a building which was designed primarily to achieve structural economy. The design of this structure is a refreshing change from the current procedure in architectural engineering. As a general rule the architect first develops a building rather thoroughly in accordance with his ideas of architecture. The engineer is then called in and invited to support the building in such spaces as the architect has not seen fit to utilize in his treatment. The resultant structure is, of course, a compromise between engineering design and architectural concession. In this instance from very necessity the building was handled, from its inception, as an engineering problem. The architectural treatment was subordinated to the engineering requirements, and, indeed, it consists of little more than a refinement and decoration of the structure.

In a larger sense, this structure has added nothing new to architectural engineering. There are no features inherently novel in the design. It is worthy of consideration, however, in that its conceptions are fundamentally sound. Two main considerations govern the design: (1) Selection of the materials best adapted to the specific requirements; and (2) refinement of engineering design to secure maximum efficiency from the materials chosen.

In a balcony of this magnitude steel framing is decidedly indicated. The large cantilever length would definitely predicate such construction even if considerations of head-room and exit clearance did not demand it. In connection with the roof, the chief desideratum was to erect a structure which would be light and which would not exert a large bending moment on the long columns that support it. Steel trusses on rollers met the requirements very satisfactorily. Throughout the remainder of the structure, the requirements were not so severe, and reinforced concrete was chosen as being adequate for the construction involved and considerably cheaper than steel. As indicative of the economy of construction attained, attention is called to several features which may be of service in similar structures.

In the wall footings, the height of the wall below the first floor was used as a moment arm in the continuous wall footings. This procedure is in accordance with the probable stress distribution and is highly efficient.

The rigid frame construction in the Banquet Hall is the most economical type of structure and has the additional merit of producing a very graceful architectural effect. Unfortunately, statically indeterminate structures are

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† Received by the Secretary April 20, 1928.

comparatively little used in the United States. There is a mistaken feeling that this type of construction involves laborious and protracted engineering calculations. In some instances this, of course, is true. Literature of the rigid frame has been developed to such an extent, however, that, in general, a rigid frame design is as simple as a statically determinate problem. More serious consideration should be given to this form of construction since it is almost invariably highly economical and pleasing in appearance.

The cantilever purlin system is a construction much in vogue in Europe. There seems to be no good reason why its use in this country should not become more general. As against a bending moment of $\frac{W\,L}{8}$, the cantilever system can be arranged to secure a moment of $\frac{W\,L}{10.45}$ in the end spans and $\frac{W\,L}{16}$ in all interior spans. This is a saving not to be regarded lightly. The system is applicable to floor construction and has been used by the writer in that connection with notable economy. It is particularly applicable to long-span floor construction, where its use not only achieves economy, but also decreases the depth of a floor system to a minimum not otherwise attainable.

R. McC. Beanfield,* Assoc. M. Am. Soc. C. E. (by letter).†—Considering the scope and valuation of the building industry comparatively little descriptive and practical data, relative to the actual design and construction of structural frames, particularly those involving statically indeterminate problems, has been published. Many theories have been advanced and discussions written about the design of building structures, but practical descriptive matter of value, covering the actual design, construction, and tests of such structures, is woefully lacking.

With this thought in mind the writer submitted his paper with the hope that, in the solution of his problems (some of which were rather unusual), he would contribute some practical data of value to those interested in structural design and economies of building construction.

Mr. Waller‡ calls attention to the fact that the building was treated, from its inception, as an engineering problem to which the architectural treatment was more or less subordinated. By the very nature of the structure, with its unusual long-span elements, it consisted, to a large extent, of purely engineering problems. On the other hand, the esthetic treatment was not without its difficulties, and these were solved efficiently and economically by the architect. The desired efficiency was produced by 100% co-operation between engineer and architect, and resulted in many economies in the actual construction of the building.

Is it not really a fact that many modern buildings express false conceptions of architectural beauty—particularly of strength and stability—by the illogical use of decorated veneers, arches, and beams that do not seem to have substantial supports, and by other deceptive motifs belying strength and supporting beauty?

* Structural and Mech. Engr., Los Angeles, Calif.

† Received by the Secretary, July 18, 1928.

‡ See p. 2123.

In the "Shrine Civic Auditorium" the architect expressed the large structural units which, for the most part, were exposed, as elements of architectural beauty. An arch is always more pleasing than a straight soffit beam. Why not use more concrete arches as real supports, thereby obtaining 100% use of the concrete, whereas only about 38% is actually utilized in a reinforced concrete beam? Why resort to the use of plaster when the natural surface of the concrete can be treated, or decorated, with excellent results and more economically, as was done in many of the structural elements of this building?

Special attention is directed to Mr. Waller's statements,* namely,

"* * * (1) Selection of the materials best adapted to the specific requirements; and (2) refinement of engineering design to secure maximum efficiency from the materials chosen."

It is an exception rather than a rule, that the structural engineer is not permitted to use his judgment in the selection of the most efficient materials for a specific purpose. The arbitrary policy of dictating that this or that material must be used throughout the structure tends toward considerable waste of money.

Without the use of the elastic theory it would have been practically impossible to design most of the large structural units with any degree of safety or economy, because many of the structural elements were statically indeterminate in varying degrees. Reinforced concrete, owing to its monolithic characteristics and the fixed conditions and continuity at the joints, involves more or less statically indeterminate problems, which should be solved by the application of proper methods based on the elastic theory. It will avail nothing to ignore the stern fact that arbitrary and illogical rules, as adopted in most municipal building codes, will serve as a panacea for the design of such structures. There is ample information and precedent now available to those who will interest themselves in modern statics.

Mr. Waller* emphasizes the methods of designing continuous wall footings wherein considerable economy is obtained by locating the tension steel high up in the wall above the footing proper. Mention is also made of the economy involved in the cantilevered purlin system, which can likewise be applied to floor systems.

Mr. Noetzli's discussion† has supplemented the paper with several interesting features of the structural design, particularly the proscenium arch. Enlarging on his suggestions relative to extensometer readings, the writer strongly advocates the measurements of deformations in large and important structural elements of a building, especially of statically indeterminate reinforced concrete frames. For example, it would be enlightening to know the magnitude and effect of secondary stresses in long-span reinforced concrete trusses, several of which have been constructed within the past five years. The measurement of stresses in rigid frames, verifying theoretical assumptions, would do much to create increased confidence in and the more extensive use of such structures.

* See p. 2123.

† *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1580.

Relative to extensometer readings Mr. Belzner states:*

"When the pressure was always placed directly over the contact points and firmly held against the member to be measured, there could not possibly be any spring of the various parts in the strain-gauge, causing variations in the readings."

The personal equation of the operator has much to do with the accuracy of strain-gauge readings. Applying equal, vertical pressures on two points 8 in. apart, sometimes in difficult positions, by one man, is easier said than done. Unless the pressures are exactly vertical over the contact points a variation in the deformation read is certain. For this reason an average of three measurements was taken.

To read deformations by extensometers accurately requires considerable technique and varies for different types of strain-gauges. More published data on the technique of strain-gauge measurements would be of considerable assistance to the profession.

Mr. Floris† submits a commendable discussion relative to the solution of the rigid frames supporting the Banquet Hall roof. His suggestions relative to the use of hinges and the avoidance of fixity by brackets at the column joints, at the first and second-story girders, in order to reduce to a minimum the statical indeterminate conditions, are noteworthy and lead to a definite determination of all stresses in the frame. On the other hand, the value of the rigidity of the structure, as a whole, must not be overlooked for conditions that are produced by seismic disturbances. With slight variations in the seismic oscillations a series of loosely jointed beams and columns would act as battering rams, resulting in much probable damage to the entire structure.

The writer's analysis of the stresses in these frames assumed the main columns to be hinged at the second-floor level, with hinges at the outer supports of the roof girders, thus obtaining a frame statically indeterminate in one degree—that of the horizontal reactions at the column hinges. Other parts of the frame, below the column hinges, were designed on the basis of relative rigidities.

The solution of the three-hinged frame, as suggested by Mr. Floris, is quite novel and instructive. The method of analysis is given in detail and contains general formulas for the most practical loading conditions.

That the writer has been repaid for his efforts is reflected in the assistance he has been able to render others desiring data involved in the solution of some of the problems presented in his paper. Furthermore, the reprinting of the writer's paper in part by other technical publications, interested in the building industry, was a source of considerable satisfaction, as it appeared to arouse interest on the part of non-members of the Society.

The writer is grateful to those who have so generously contributed discussions tending to increase the value of the paper by emphasizing certain features well worthy of special notice.

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1579.

† See p. 2119.

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PAPERS AND DISCUSSIONS

THE DESIGN OF AIRPLANE WING-BEAMS

Discussion*

By Alfred S. Niles, Assoc. M. Am. Soc. C. E.

Alfred S. Niles,† Assoc. M. Am. Soc. C. E. (by letter).‡—The formulas proposed by the author for computing the bending moments and deflections of beams subjected to combined axial and transverse loads appear at first to be too complicated for use in practical design. Four years' experience by the military services and the commercial designers has shown, however, that such doubts are unfounded. Once a designer has become accustomed to these formulas, he finds that they are easier to apply than any of the approximate methods formerly used, that gave results which were at all satisfactory in airplane stress analysis.

In 1922, when the writer assigned to the author the problem of developing a practical system of computing the bending moments in a beam subjected to combined axial and transverse loads, the methods used for solving problems of this class were very unsatisfactory. The Navy used the Berry formulas which had been obtained from the English, but these formulas had two serious defects. The most serious was that the formulas applied only to the case of a beam with a uniformly distributed transverse load, but other types of transverse load were constantly occurring in design. The other was that the tables of coefficients for use in the three-moment equation gave the values of these coefficients for different values of the quantity corresponding to $\frac{L}{j}$ expressed in degrees rather than as a non-dimensional number. This meant that $\frac{L}{j}$ and $\frac{x}{j}$ had to be constantly changed from numbers or "radians" to "degrees"

* This discussion (of the paper by Joseph S. Newell, Jun. Am. Soc. C. E., published in March, 1928, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, June 6, 1928.

and back again, an operation that was not only exasperating to the computer, but resulted in an unavoidable loss of precision in the computations. The Army had never adopted the Berry method, partly on account of these defects and partly because of lack of faith in its theoretical basis, but used various approximate methods that were also unsatisfactory.

After a study of the Berry method and the parallel method of Müller-Breslau, and comparing their results with those of the approximate methods used by the Army and others that had been proposed, Mr. Newell decided that, as the Berry and Müller-Breslau methods were basically extensions of the ordinary beam theory, their use would be greatly facilitated if the formulas were modified to agree with the system of nomenclature in common use by airplane designers in this country, and tables of functions of $\frac{L}{j}$ and $\frac{x}{j}$ as numbers were substituted for the tables of functions of these quantities expressed in "degrees".

Before completing his work on the problem, the author not only developed the formulas for the cases covered by Berry and Müller-Breslau, but also for some additional cases of importance in airplane design, some of which are included in the paper. In addition, tables of the coefficients (Table 8*) were compiled, as well as tables of the sines, cosines, and tangents of numbers from zero to 3.50, which, unfortunately, were omitted from the paper. It was the existence of these tables making it unnecessary to carry through any transformations from numbers to "degrees" in using the formulas that, more than anything else, made the method acceptable to the practicing airplane designer.

As soon as the formulas and tables of functions had been completed, and the experimental work described in Part III† had justified their use in design, they were published in pamphlet form and supplied to the designers of airplanes for the Army and Navy. Although their use was strongly encouraged by the Army, both that Service and the Navy continued to permit the Berry method to be used. It was not long, however, before the commercial designers decided that the author's formulas were the simpler and began to use them exclusively, not only for Army and Navy work, but also for their commercial designs. At present, they are in use by practically all airplane designers in this country, and the cry of the industry is not for simpler methods of computation, but for more accessible publications in which they can be found.

The work by the Forest Products Laboratory that is described in Part II,‡ was done independently from that of the author. Originally, it suffered from the defect that in order to apply the method proposed for determining the ultimate load for a beam it was necessary to know the magnitude of the secondary bending, *Py*. The author's work then suffered from the complementary defect that while the formulas could be used to compute the secondary bending, it gave no indications as to when the ultimate load, as distinguished from the buckling load, was reached. By combining the results of the two

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 781, *et seq.*

† *Loc. cit.*, p. 769.

‡ *Loc. cit.*, p. 754.

lines of work, a complete and satisfactory plan for computing the ultimate load for an airplane wing was obtained.

The writer is conscious that these notes on the history of the work leading to the author's paper may be of little interest to engineers in lines other than aeronautics. It should be realized, however, that the formulas of the paper are not impractical theory, but are those used in the stress analysis of most of the airplanes in the air to-day.

Secondary Shear.—For beams subjected to transverse loads only:

$$E\,I\,y = \int E\,I\,i\,d\,x = \int\!\!\int M\,d\,x\,d\,x = \int\!\!\int\!\!\int S\,d\,x\,d\,x\,d\,x$$
$$= \int\!\!\int\!\!\int\!\!\int w\,d\,x\,d\,x\,d\,x\,d\,x \qquad (46)$$

In developing his formulas for beams with axial as well as transverse loads, the author assumed the relationships between moment, slope, and deflection to be those expressed in Equation (46), but he has not stated whether those between moment, shear, and load also exist in this case. This depends primarily on the definitions used for the shear and the transverse load.

In most beam computations the shear at any point along the beam axis is assumed to be the shear on a section through that point and perpendicular to the location of the beam axis before the loads were applied. For purposes of web design it would be more correct to compute the shear on sections perpendicular to tangents to the elastic curve of the beam at the points in question. Since the angle between such a tangent and the original location of the beam axis (the slope, i) is always assumed to be very small, the usual practice is justified, first, because the error is both negligible and on the safe side; and, second, because the computations are greatly simplified.

In the cases of beams subjected to axial loads that are large in comparison to the transverse loads (and most airplane wing-beams fall into this class), the error involved in the usual method of computing the shear is no longer negligible, and for proper design the shear on sections perpendicular to the elastic curve must be computed. In a study of airplane wing-beam tests made to determine the constants to be used in the design of webs for box-beams,* Mr. Roy A. Miller found cases in which the shear on sections perpendicular to the elastic curve was as much as 20% greater than that on sections perpendicular to the original location of the beam axis. His results also showed that this additional shear had to be taken into account in design. Mr. Miller's studies were confined to wood box-beams, but his conclusions regarding the importance of the secondary shear are equally applicable to the design of the web members of metal trusses used as wing-beams.

The magnitude of the total shear on a section perpendicular to the elastic curve of a beam can easily be computed. In Fig. 23, $C\,F$ is the elastic curve of the beam; $C\,D$, a tangent to $C\,F$ at C; $D\,E$, the original location of the beam axis before loading; and $A\,B$, a section through the beam at C and perpendicular to $D\,C$. In the same diagram the forces acting on Section $A\,B$ from the portion of the beam to the left are represented by P and S, parallel

* U. S. Air Corps *Information Circular 516*, "The Design of Plywood Webs for Box Beams."

and perpendicular, respectively, to the original axis, $D E$. P is what is usually thought of as the axial load and S, the shear as usually computed.

The normal force on Section $A B$ is then obviously equal to $P \cos i + S \sin i$, and the shear force is $S \cos i - P \sin i$. If the slope, i, is assumed to be small, $\cos i = 1.00$ and $\sin i = i$, practically. Then, the normal force on Section $A B$ will be $P' = P + S\,i$, and the shear will be $S' = S - P\,i$.

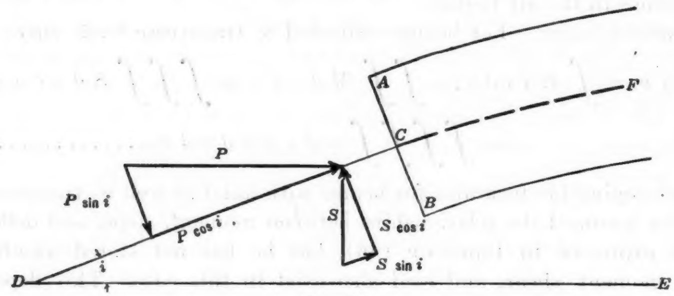

FIG. 23.

From the author's paper it is obvious that,

$$M = M_0 - P\,y \quad \text{..........................(47)}$$

in which, M is the total bending moment; M_0, the bending moment due to transverse loads alone; P, the load parallel to the original axis of the beam, positive when in compression, and assumed to be constant; and, y, the deflection (positive when upward).

Differentiating Equation (47) with respect to x,

$$\frac{d\,M}{d\,x} = \frac{d\,M_0}{d\,x} - P\,\frac{d\,y}{d\,x} \quad \text{..........................(48)}$$

From Equation (46), however, $\frac{d\,M_0}{d\,x} = S$, the shear on a section perpendicular to the original axis of the beam, and $\frac{d\,y}{d\,x} = i$, the slope of the elastic curve. Hence Equation (48) may be written,

$$\frac{d\,M}{d\,x} = S - P\,i \quad \text{..........................(49)}$$

or the derivative of the total bending moment with respect to x is equal to the shear on a section perpendicular to the elastic curve of the beam. Thus, Equation (46) is seen to express the relationship between the most important shears and the total bending moments on a beam subjected to axial compression. Similarly, the last part of the equation can be shown to express the relationship between this shear and what might be called the true transverse load on the deformed beam.

Effect of Shear Deformation on Bending Moments.—In most computations of beam deflections, the deflection due to shear is assumed to be negligible, and the author has followed this practice in the paper. In the case of the axially loaded beam, as the total bending moments are so largely due to interaction of the axial load and the deflection, it is of interest to see whether it is reasonable to retain this assumption.

The relative deflection due to shear of any two sections at a distance, dx, apart will be, $dy = K\ S\ dx$, in which, S is the shear at the section and K, a constant depending on the shape of the cross-section and the material of the beam. This may also be written,

$$\frac{d\,y}{d\,x} = \frac{K\,d\,M}{d\,x} \qquad (50)$$

it being understood that y is the deflection due to shear only.

In order to study the effect of neglecting this shear deflection in practical computations of total bending moments, a simple case—that of a pin-ended strut with axial compression and a uniformly distributed transverse load—will be considered.

Let y = total deflection;
y_b = deflection due to bending; and,
y_s = deflection due to shear.

From the beam theory,

$$M = E\,I\,\frac{d^2\,y_b}{d\,x} \qquad (51)$$

and from Equation (50),

$$\frac{d\,y_s}{d\,x} = \frac{K\,d\,M}{d\,x}, \text{ or } y_s = K\,M + C \qquad (52)$$

$$M = -\frac{w\,L\,x}{2} + \frac{w\,x^2}{2} - P\,y_b - P\,y_s \qquad (53)$$

Then,

$$E\,I\,\frac{d^2\,y_b}{d\,x^2} + P\,y_b + P\,K\,M + P\,C = -\frac{w\,L\,x}{2} + \frac{w\,x^2}{2}$$

$$E\,I\,\frac{d^2\,y_b}{d\,x^2} + P\,y_b + P\,K\,E\,I\,\frac{d^2\,y_b}{d\,x^2} + P\,C = -\frac{w\,L\,x}{2} + \frac{w\,x^2}{2}$$

Differentiating twice with respect to x,

$$(1 + P\,K)\,\frac{d^2\,M}{d\,x^2} + \frac{P}{E\,I}\,M = w \qquad (54)$$

Let $j^2 = \frac{E\,I}{P}$, $Q^2 = 1 + P\,K$, and $j\,Q = J$; then Equation (54) becomes,

$$\frac{d^2\,M}{d\,x^2} + \frac{M}{J^2} = \frac{w}{Q^2} \qquad (55)$$

Whence,

$$M = C_1 \sin\frac{x}{j} + C_2 \cos\frac{x}{j} + w\,j^2 \qquad (56)$$

Evaluating C_1 and C_2 from the fact that $M = 0$ when $x = 0$ and when $x = L$,

$$C_2 = -\,wj^2$$

and,

$$C_1 = \frac{w\,j^2\left(\cos\frac{L}{J} - 1\right)}{\sin\frac{L}{J}}$$

whence,

$$M = w\,j^2 \frac{\left(\cos\frac{L}{J} - 1\right)\sin\frac{x}{J}}{\sin\frac{L}{J}} - \left(1 - \cos\frac{x}{J}\right) \ldots\ldots (57)$$

The maximum moment will be at the mid-point of the strut where $x = \frac{L}{2}$.

For that condition Equation (57) becomes,

$$M_{\text{max.}} = w\,j^2 \left(1 - \sec\frac{L}{2J}\right) \ldots\ldots (58)$$

It may be noted that the difference between these formulas (which allow for the effect of the shear deflection) and those that neglect it, is that j is replaced by J in those places where it occurs as part of a trigonometric function of $\frac{x}{j}$, or $\frac{L}{j}$, but not in the term, $w\,j^2$.

As the shear deflection of a part of a beam over which the shear is positive is such that the right-hand end will be lower than the left-hand end, the constant, K, will always be negative regardless of the shape of the cross-section and material of the beam. Then, as $J^2 = j^2\,(1 + P\,K)$, J will always be less than j, and $\frac{L}{J}$ greater than $\frac{L}{j}$. In other words, in any given case, a member is nearer to buckling failure than is indicated by the formulas neglecting shear.

The Forest Products Laboratory has made studies to determine the values of K for sections suitable for use in airplane wing-beams.* By using the values of K obtained in that study it becomes possible to compute the magnitude of the error involved in neglecting shear deformations in computing total bending moments and deflections.

The percentage error due to the neglect of the shear deformations cannot be determined for the general case, although it can be done for any specific case. To obtain an approximate idea as to its probable magnitude, a single case will be analyzed. The pin-ended strut used in the development of Equation (58) will be assumed to be 100 in. long, 1.11 $\left(\frac{10}{9}\right)$ in. wide and 3 in. high. Let E = 1 600 000 lb. per sq. in., and the shearing modulus, G = 108 000 lb. per sq. in. The axial load will be taken as 1 000 lb. and the transverse load as 1.00 lb. per in.

Then,
$$\begin{aligned} I &= 2.50 \text{ in.}^4; \\ E\,I &= 4\,000\,000 \text{ in. units}; \\ j^2 &= 2\,500; \\ j &= 50 \text{ in.; and} \\ \frac{L}{j} &= 2.00. \end{aligned}$$

* National Advisory Committee for Aeronautics, *Technical Report No. 180*, "Deflection of Beams with Special Reference to Shear Deformations," by J. A. Newlin, M. Am. Soc. C. E., and G. W. Trayer.

The constant, K, for a rectangular section* is $-\frac{1.2}{A\,G}$, in which, A is the area of the cross-section. For this case then,

$$E\,I\,K = -\frac{4\,000\,000 \times 1.2}{3.33 \times 108\,000} = -13.33$$

$$J^2 = j^2 + E\,I\,K = 2\,500 - 13.33 = 2\,486.7$$

$$J = 49.8665 \text{ and } \frac{L}{J} = 2.00535$$

The maximum error in bending moment due to neglect of the shear deflection will be located at the mid-point of the strut, where the ratio of error will be,

$$\frac{1 - \sec\dfrac{L}{2\,J}}{1 - \sec\dfrac{L}{2\,j}} - 1$$

From the determined values of $\frac{L}{J}$ and $\frac{L}{j}$, the value of $\sec\frac{L}{2\,J} = 1.8586$ and $\sec\frac{L}{2\,j} = 1.8508$; and the error is $\frac{78}{8\,508} = 0.0092$, or a little less than 1 per cent.

If the beam had been of I- or box-section, or if $\frac{L}{j}$ and $\frac{L}{J}$ had been larger, the percentage of error would have been greater. On the other hand, if, instead of using the true value for the modulus of elasticity for E, a somewhat smaller figure had been used, the value of $\frac{L}{j}$ would have been larger. If the correction to E were large enough to make $\frac{L}{j}$, as computed, larger than $\frac{L}{J}$ would be if computed from the true value of E, any error in the bending moments will be on the safe side.

In practice the value of E used in computations is about 10% less than the true value in order to obtain this result, thus making an allowance for shear deflection in the computations of flexural deflection. This is an arbitrary method of making the correction; but it is simple, and works satisfactorily. As a consequence, the formulas for bending moment, such as those just derived by the writer, are of no practical use, and the study outlined is of interest primarily to show that the current practice of using the author's formulas, in the derivation of which shear deflection was neglected, is justified.

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PAPERS AND DISCUSSIONS

FLOOD CONTROL ON THE RIVER PO IN ITALY

Discussion*

By Messrs. John P. Hogan, William B. Gregory, John R. Slattery, Arthur P. Davis, and G. de Thierry.

John P. Hogan,† M. Am. Soc. C. E.—The author has called attention to the fact that there really is a flood-control proposition that is being studied in a scientific manner. This is particularly appropriate at a time when, during the past year or year and one-half, engineers have been attempting to handle a flood-control proposition partly on a political basis and partly on a sentimental basis. The speaker refers, of course, to the Mississippi River.

It would seem that in order to provide flood protection, the first and foremost thing would be the necessity of determining what the maximum flood might be. Allen Hazen, M. Am. Soc. C. E., has given a very great contribution to this subject‡ which, of course, is as applicable to flood control as it is to river regulation. Nevertheless, the law of probabilities will never foretell the maximum flood that will occur. It will only predict the probability of the occurrence of a flood. It may be known that a certain flood will occur once in 100 years, once in 20 years, or once in 10 years, but it will not be possible to predict that perhaps to-morrow, or next year, there is going to be the flood which is due once in 10 000 years. It would seem to be axiomatic that absolute and positive flood protection by completely confining the waters of a river within a narrow channel is not economically feasible, and there is considerable doubt as to whether it is practically possible.

Without any further study and with very indefinite knowledge on the subject, efforts have been made to pass a blanket resolution providing inde-

* Discussion of the paper by John R. Freeman, Past-President, Am. Soc. C. E., continued from August, 1928, *Proceedings*.

† (Parsons, Klapp, Brinckerhoff and Douglas), New York, N. Y.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

finite sums of money to control the Mississippi River. There has been absolutely no effort whatever made to accompany the plans for control with any economic study, to determine whether such a regulation is advisable. It certainly is evident that it does not pay to spend $1 000 000 000 to save $100 000 000.

It would also seem evident that the river must have a channel with certain marginal districts of land subject to periodic flooding, as Mr. Freeman has shown in regard to the Arno. It would seem that the first process in determining a reasonable plan of regulation for the Mississippi River would be an estimate of the present value of the land affected and its prospective value. A determination could be made, on that basis, of the present and future methods that should be used either to reclaim or preserve from flood at the present time the maximum acreage of land which might economically be reclaimed, and to provide for certain future measures that could be applied if and when the reclaimed land becomes sufficiently valuable to warrant it. Further large expenditures might be justified on the broad grounds of public welfare, but the speaker believes that the economic survey should be the first step.

Land owners have been crowding into the lowlands for the past 50 or 100 years as land became more valuable. They have been filling up the channels which were formerly open to the river in time of flood. It would seem that it is about time, as far as a river like the Mississippi is concerned, to take stock, and to find what should be reclaimed and preserved either in the present or in the future.

In presenting this view of flood control on the River Po in Italy, Mr. Freeman has done a real service.

William B. Gregory,* M. Am. Soc. C. E.—The speaker is very much pleased with Mr. Freeman's paper. It is a splendid example of the ways in which hydraulic information should be gathered and applied to a problem such as that of the Po.

There are many people in New Orleans, La., who think because they have lived on the banks of the Mississippi River and have seen it all their lives, and because their fathers saw it all their lives, that they know all about it; and one of the most common fallacies is the belief that the bed of the river is rising. They have seen the levees rise as the years go by; in fact, in the thirty years that the speaker has lived in New Orleans the levees in the front of the city have been raised about 6 ft.; and, of course, people who are not acquainted with hydraulic laws jump to the conclusion that the bed of the river is rising, which is not true. What Mr. Freeman has emphasized in connection with the Po, is also true of the Mississippi; namely, that it is not scouring out a deeper bed, as was hoped years ago.

The Mississippi River Commission began with the idea that confining the river within levees would scour out a deeper channel and improve that channel. In an address at Memphis, Tenn., C. McD. Townsend, M. Am. Soc.

* Irrig. Engr., U. S. Dept of Agriculture; Prof. of Experimental Eng., Tulane Univ. of Louisiana, New Orleans, La.

C. E., a few years ago, stated that while that might be true extending over a long period of time, it would be of more interest to the people who will occupy the valley in the Twenty-fifth Century than to those who live there now.

Incidentally, Colonel Townsend has stated* that the Mississippi River Commission had every bit of information that is necessary for the control of that river, and all that needs to be done for it in the future.

JOHN R. SLATTERY,† M. AM. SOC. C. E.—It seems always of great importance to bring out the fact that the bed of the Po and the bed of the Mississippi have not risen. At least, if the bed of the Mississippi has risen, it has been at so slow a rate that as far as the engineers of the present day are concerned it need not be considered. That misleading observation of the old priest traveling along the Po has caused more sorrow and grief to engineers engaged in flood control on alluvial streams than almost any other fallacy.

A few years ago, C. McD. Townsend, M. Am. Soc. C. E., made a most exhaustive study of the Italian records. He did not read or speak the language before that time, but he studied enough Italian to delve into the records of the Po River, and his paper on that subject was one of the most conclusive that has been written. The paper was not published; it is in the archives of the Mississippi River Commission.

Spur-dikes or retards have been tried on many rivers in this country as well as in Europe. The great trouble with that system of improvement seems to be that, while something is accomplished, the end sought is never fully realized. A system of spur-dikes to produce a channel 9 ft. deep may be designed. The system is completed, some improvement results, but the desired 9-ft. channel is rarely, if ever, obtained. An elaborate system of dikes of that character has been under construction above Cairo, Ill., on the Mississippi River, for a number of years and has produced a marked improvement.

In early years some attempt was made at Lake Providence, or in the Providence Reach and the Plum Point Reach, at river control, and it was more or less successful. It was abandoned because of the great expense and because through the development of dredging it was unquestionably cheaper to maintain the necessary depth by this method than by a system of permanent structures.

The flood-fighting methods on the Po, mentioned by Mr. Freemen,‡ are quite similar to those used on the Mississippi. In fact, those officers who have been engaged on the Mississippi are made to study quite thoroughly the history of the Po as well as of all other similar rivers. Many plans are thought to be new when as a matter of fact they have been tried and failed. The Mississippi River Commission has been liberal in providing money for the investigation and trial of really new ideas, but has properly set its face against wasting money in investigation of plans already thoroughly tried out without satisfactory results.

One of the great difficulties on the Mississippi up until the present has been the divided control. In the 1916 flood the speaker saw the gauge at Cairo

* *The Military Engineer,* March–April, 1928, p. 93.

† Deputy Chf. Engr., Board of Transportation of the City of New York, New York, N. Y.

‡ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 971.

gradually rising, and about three weeks before the crest reached Vicksburg, Miss., it was realized that the flood that year would be the highest that had ever been known. It was evident at that time that a severe fight was on hand. It was evident, also, that certain stretches of levees would have to be built up with sand bags.

As the flood advanced, the necessity of carefully guarding the levee was realized. Most levee problems (if the levee is not overtopped), can be controlled, providing warning is received in sufficient time and providing the proper supply measures are taken at the beginning. The speaker, therefore, through the newspapers of Greenville, Vicksburg, and the small towns that published papers, urged all owners having plantations facing on the levee system to undertake the patroling of their own levees.

They all have large numbers of negroes; there is no scarcity of help. These people are practically carried through the winter season anyway by the planters for whom they work. There was immediate and violent opposition on the part of one levee official because of what he seemed to think was the usurpation of some of his authority, and it took a great deal of diplomacy before he was finally mollified and brought to co-operate in securing the proper guarding of that levee system.

Millions of sacks that year were purchased, 6 000 000 in that one district, and distributed to various warehouses along the river. The plant of the district consisted of 5 towboats and about 80 barges. Those were distributed at strategic points along the river. Barges loaded with gravel and lumber were stored in the lower reaches where the back-water up the Yazoo Valley attains almost the same height on the levee as the water on the outside.

The levee line was carefully watched; a warning was sent of a boil at this point; a sloughing bank at that point; water lapping over the levee at another point; and within a few hours it was possible to move men and supplies to any point on that river, and that was the way that the levee system was held. At three o'clock one morning a steamboat captain telephoned that the water was running over the levee a little north of Vicksburg for a distance of about 5 miles. At seven o'clock the next morning barges of gravel and sand bags were at that particular point. The back-water was so high that no material could be obtained from the levee itself. Nevertheless, if the levee were lost, it meant an extension of the flood a number of miles up the river. The actual difference between the level of the back-water and the water in the river at that point was about 5 ft. It took a week of fighting, but at the end of the first day there was a line of sacks above the flood, and that flood was followed up until it crested by adding line after line of sand bags.

Some of the methods that are now suggested for the improvement of the river have received most careful consideration in former years. One reason why they have never been adopted has been the strong local opposition to anything but levees.

The Cypress Creek Gap was closed while the speaker was in Vicksburg, or rather, it was started. It was realized that the closing of that gap would immediately raise the height of floods for some distance below the mouth of the Arkansas, because the closure cut off an enormous reservoir in Arkansas

and Louisiana. Much of the land in this region was not particularly valuable; a great deal of it was not and probably is not to-day under cultivation; a great deal of it was wooded; there were many cotton plantations so infested with boll weevil that cultivation had ceased. At the same time the local pressure to stop the overflow, which took place at a stage of 49 ft. at Arkansas City, was so great that the Mississippi Commission had to close the gap. The 1922 flood confirmed, to within 1 ft., calculations made in 1916, as to the effect, of closing that gap, on river stages at Arkansas City and vicinity.

The overflows are not always an unmitigated curse. No planter wants to see his land overflowed; but many a planter, after a crevasse has occurred, finds that the following year the increased fertility of the soil more than offsets his loss for the one year of flood. It sometimes happens that, instead of coating his plantation with a film of rich silt, sand is carried in and his plantation is ruined for years to come, so it is a gamble. There are parts of the Mississippi Valley to-day that would be greatly benefited if they could be overflowed. The soil has been worn out, and the rich plantations that once existed there cannot now pay expenses.

The Po and the Mississippi are near relatives, but the levee section on the Mississippi is quite different. There has long been consideration given to the desirability of putting a road on these levees. The river slopes on the Mississippi levees are 1 on 3, or, in certain soils, 1 on 4. The top width is 8 ft. The slope down to the banquette on the land side is 1 on 3. The banquette is from 8 to 12 ft. below the crest, and varies in width from 20 to 40 ft., with a slope of 1 on 6. The lower slope on the land side is about 1 on 4. This heavier section is necessary, as Mr. Freeman pointed out,* because of the long period that the water stays high; and on the Mississippi those fighting the floods do not feel safe until the water has fallen about 10 ft. below the crest of the levees. Even after the water starts to fall, there is still danger of these levees failing with the reduced head through sloughing.

On the matter of cut-offs or the straightening of rivers, in this country the general rule is not to straighten a river except where streams, such as those in Florida, have a very slight slope. There, the streams are materially shortened to their great advantage. On the Mississippi, however, every effort is made to prevent cut-offs. Above Greenville, for instance, there is a long bend around a long point of land. It was 5 miles around this point and only about 2 400 ft. across it. A cut-off would have shortened the river about 4 miles.

The river began to cut channels across the point, and an attempt was made to fill them by permeable dikes. They were not successful. It was impossible to induce those channels to fill. The banks were heavily revetted along the upper side of the point, but some signs of failure were noted. It was recognized that if the neck was cut through, at least one-half the levee system for perhaps 50 miles below would go as a result of the bank-caving that would follow from the excessive currents that would exist until such time as the river re-adjusted its slope.

An earthen dike about 3 or 4 miles long was built from the levee system out to near the point. Straightening the Mississippi is not to be thought of unless the banks and bottom for many miles above and below cut-offs can be

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 976.

adequately protected against scour. To any one familiar with conditions on the Mississippi above Baton Rouge, La., the problem of preparing the bed to carry the river safely at a greater slope, comes within the realm of economical impossibilities.

ARTHUR P. DAVIS,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).†—The River Po presents unusual opportunities for the study of river hydraulics, not only on account of its combination of physical conditions, but also due to the long history of its use and modification by human agencies. Mr. Freeman's paper is of great interest and shows industrious research into the history and literature of this river.

The typical large river system, of which many examples can be found in the world, is a collection of drainage from high or mountainous regions, which flows to the sea, eroding its channels in the upper reaches and depositing the eroded material at its mouth. In the course of hundreds of thousands, or perhaps millions, of years, the basin is lowered some hundreds or thousands of feet by the influence of the waters of this river system. Where the climate is humid and the rainfall considerable, the hilltops are weathered and disintegrated by the action of the rain and frost, and the tendency is to produce a rounded topography such as that seen in the Appalachian Mountains, and in the main in the Rocky Mountain System as well. If the river through its course carries its load of water and sediment through an arid region where the surface weathering is slight and where the declivity of the river permits rapid erosion, deep and narrow canyons may be produced, as in the Colorado System of the Southwest.

Throughout the history of such a river system, the forces of Nature are actively at work degrading the topography toward the condition known by geologists as "base level", and, if not interfered with, the entire basin would, with lapse of time, be reduced to a "base level" or a region so flat that any movement of water thereon would be so sluggish as to produce no erosion. Limited areas of such base levels occur near the mouth of every large deltaic stream, such as Southern Louisiana on the Mississippi, and the great delta of peat land area near the junction of the Sacramento and San Joaquin Rivers in California.

Wherever the river has declivity enough to produce erosive velocities, it continues to erode its channel until this is reduced to an elevation such that the fall to the sea is insufficient to produce such velocity. The slope that is necessary varies, of course, with the quantity of water flowing, the larger quantities producing higher velocities, so that there is a critical stage in some part of such rivers where the channel has a tendency to erode at high stages and to refill at low stages.

The work of the river through the centuries, bringing down its load of sediment, builds a larger and larger delta at its mouth through which the river meanders a sluggish course at ordinary stages, depositing the load that it has brought from the mountains. Many interesting examples of this may be cited.

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† Received by the Secretary, June 25, 1928.

The Yangtze-Kiang in China has advanced its mouth thirty miles into the Yellow Sea by progressive delta building during historic times. The great deltas of the Mississippi and Sacramento Rivers are well known in detail. The Colorado built a delta at its mouth extending from the California line entirely across the Gulf of California, severing the head of the Gulf from the main body and forming thereby an inland sea which evaporated and formed the Imperial Valley lying below sea level, with a strongly saline lake in the bottom. This delta of the Colorado River has been built to an elevation of 40 ft. above sea level at its summit and reaches north and south nearly 100 miles.

As the mouth of such a river advances into the sea, the increased length causes a reduction of the gradient of the stream which is emphasized also by the reduction of the elevations higher up by erosion. This decrease of gradient deprives the river of part of its carrying power and causes the deposit of a portion of its sediment farther and farther up stream; so that for some distance up stream from its mouth there is a gradual tendency to the upbuilding of its channel. As this upbuilding progresses, the river bed rises and creates a tendency to overflow which is characteristic of every such river throughout its lower course. As the river begins to overflow during the flood, the water overflowing the banks is checked in velocity where it encounters the obstruction of vegetation and shallow depths, and immediately deposits the coarser materials it is carrying. This tends to build up the immediate banks of the river, while the partly clarified water spreads out over the valley and deposits a thinner layer of sediment. Thus is formed the characteristic topography of a river channel as it approaches the base level condition, where the banks of the river are higher than the grounds immediately back of them and there is a tendency for the river to run on a ridge as illustrated in the paper.*

This unstable condition cannot continue, however, and some time at flood the river breaks the banks and flows through the lower country in a new channel where, in time, the same process is repeated; and thus the river swings back and forth in the course of centuries and builds up a broad alluvial valley to an extent sufficient to maintain such velocities as will carry a portion of its sediment to the sea. Thus, the combination of conditions involves the processes, (*a*) the degrading influence throughout the mountainous regions of the stream near its head-waters; (*b*) the erosion of its channel through the upper course; (*c*) the building of a broad alluvial valley through its lower course; and (*d*) the extension of the delta at its mouth into the sea.

The writer visited the Po Valley in 1911 and examined its largest irrigation system, and by these observations and the impressions produced by the literature on the subject, concluded that the Po River was typical of the conditions described. It is probably one of the best types to be found, rising as it does in the lofty Alps and Apennines with their high precipitation and heavy slopes, and the occurence of wide valleys with deep alluvial soil throughout its lower courses, with a large delta at its multiple mouths characteristic of such deltas.

As the Po River still comes from the high Alps and Apennines, affording abundant precipitation and abundant gradient, erosion of the channels through

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, Fig. 13, p. 992.

the mountainous regions must still be in active progress, and this material must be deposited either at the mouth of the river, or in the valley, or both.

It is interesting therefore to find that, in the "Introduction",* the author makes the following statement:

"Flood heights certainly have increased, but the best data available show that meanwhile the bed of the main channel has not been raised."

This statement is doubtless based on the report which the author quotes† from the Ministry of Public Works, as follows:

"As seen from the said studies of the Hydrographic office of the Po and from the hydrometric observations of the Po for a century, it is evident that the bed of the river presents no tendency to progressive rising."

This conclusion, in view of the surrounding facts, is interesting, for if no sediment is deposited in the river bed, it must all be carried to its mouth and deposited in the Adriatic where it would lengthen the course of the river and progressively reduce its gradient.

The confinement of the river between dikes would prevent the deposit of sediment over the floor of the valley and confine its deposit either to the river bed or to the delta in the Adriatic and would actually accelerate the growth of the delta and the consequent rise of the river bed. This tendency could be postponed and even for a time it could be reversed by straightening the river channel, which would reduce its effective length and thereby increase its gradient; and this, in turn, would increase its velocity and prevent deposits in the river bed. If such were the case, the eroded material from the mountains would travel to the sea and the growth of the delta would be accelerated and, in time, would add to the length of the river sufficiently to counteract the shortening produced by straightening. This seems to be the most plausible explanation of the apparent anomaly already noted. Mr. Freeman's opinion on this explanation would be interesting and valuable.

If this artificial straightening has not occurred, what then is the explanation of the fact that the building up of the river bed, which has been proceeding "during hundreds of thousands of years by the sand and mud eroded from the mountain slopes", has recently stopped?

Whether the bed of the Po, or of the Mississippi, or of the Yellow River, or of any other deltaic stream, has been built to an elevation higher than the adjacent country, is a question only of speed and of quantity. Hydrologists should not lose sight of the fact that all such rivers are eroding material from their upper reaches, bringing it down into the lower valleys where it is all consumed in enlarging its delta, lengthening its course, or building up its valley and rived bed, or both.

If the river is confined so that it cannot deposit its sediment in the broad valley, all this sediment must then be utilized in building up the river bed or extending the delta, or both. In the long run both must occur, else what is to become of this sediment?

The Rio Grande and some other streams have varied this program temporarily due to the operations of Man. In the case of the Rio Grande, the head-waters were consumed largely in the irrigation of upper valleys and the

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 959.

† *Loc. cit.*, p. 991.

steady flow of considerable quantities of water which formerly occurred have been largely stopped. This same flow consists mostly of clear water resulting from melting snow, and the reduction of its volume and the shortening of the season of its discharge by consumption above has deprived the river of a large proportion of its carrying capacity through the middle and lower reaches. The result is that the torrential tributaries which bring their loads of sand and gravel to the valley during sudden storms, build up their deltas faster than the main stream can carry them away and the reduction of the summer flow has greatly accelerated this process. Some parts of the bed of the Rio Grande have risen during the past fifty years from 2 to 12 ft. by actual levels, which is doubtless much faster than the rate of rise before the water was diverted at various points.

The most complete plan for the control of the floods of the Lower Colorado involves not only the regulation of the river through large storage works, but the straightening of its channel in the lower course for a distance of about 100 miles above its mouth. It is believed that it will be practicable to shorten this stretch of the river 30 or 40% by confining it between straight dikes set as close together as conditions permit and preventing the river from meandering. This, of course, will add greatly to the effective gradient of this section of the river and will cause it, for a long time to come, to erode its channel and thus remove the tendency to overflow. This effect, however, can be only temporary and will eventually be overcome by the deposits in the delta, the lengthening of the channel, and the loss of gradient resulting therefrom.

Such operations, of course, would result in a condition in which the river in this section would "present no tendency to a progressive rising", as reported by the Hydrographic Office of the Po. This would not set aside the general principle that these alluvial streams have been for thousands of years, and will be in the future, progressively building up their channels and their valleys and extending their deltas with the sediment brought down from the mountains; nor the further fact that if the stream is prevented by dikes from depositing this sediment over the wide valley, it will greatly increase the amount that must be devoted to extending the delta and building up the river channel, unless the sediment is stored above, which is not the case on the Po.

These tendencies may be modified by the storage of sediment in reservoirs near the head-waters; by the diversion of water for irrigation; and by the straightening of the channels; or, perhaps, by some other operations of Man. The effect of these operations is necessarily temporary, geologically speaking, and eventually the time must come when the river will resume building up its bed.

It would be interesting to know just what operations of Man have modified these natural tendencies in the Po Valley, and how permanent they may be.

G. de Thierry,* Esq. (by letter).†—This paper gives an excellent picture of the development of the regulation of a river on the banks of which the cradle of hydraulics has stood.

On the Po, as on all other rivers, the first works erected by the hand of Man were for the purpose of protection against the dangers incident to high

* Berlin, Germany.

† Received by the Secretary, June 27, 1928.

water. The attempt to protect as much of the adjoining land as possible from floods has led, on the Po as well as elsewhere, to a narrowing of the river channel which doubtless has constituted in places an obstacle to the free run-off of high water. The attempt to remove these obstacles subsequently has met with opposition which has been the stronger, the further the utilization of the land behind the levees has progressed. Such levees can naturally occasion a rise in high-water levels, so that frequently erroneous conclusions regarding the raising of the river bed are drawn from this phenomenon. Particularly valuable is the clear statement* that neither on the Po nor on the Yellow River in China can any raising of the river bed be traced to the effect of levees.

There is only a single particularly characteristic case of the raising of the bed of a river known to the writer. For decades it has been known that on the section of the Rhine above the Bodensee, opposite the Principality of Lichtenstein, a rise in the river bed amounting to 2 cm. yearly exists. The Rhine is diked on both sides along this section; but it would be a serious error to attribute the rise in the river bed to these parallel levees. The building of control structures along the mountain streams above this section of the Rhine is still far behind what it should be because of the very considerable cost involved. Consequently, after heavy rainfalls, or a sudden melting of snow, these mountain streams bring down enormous quantities of gravel which are in continual motion and which the river cannot digest. It is quite possible that the effect of the cut-offs and the new outlet into the Bodensee will extend far enough up stream to prevent the deposit of this gravel. In any event it will be a long time before the effect in this part of the river will be demonstrable. The creation of a new outlet into the Baltic Sea has produced a far-reaching reduction in the level of the highest water on the lower reaches of the Vistula River. The increase in the velocity involved in the shortening of the stream has brought with it a lowering of the river bed, so that on the Rhine above the Bodensee a similar effect is to be expected.

However, the Rhine below Basel, Switzerland, has proved that a long period of time must elapse before a verdict can be given. A century ago, at the instigation of Oberst Tulla, numerous cut-offs were undertaken between Basel and the Hesse-Prussian border, reducing the length of the river to 81 km. Enormous quantities of gravel were thereby set in motion. On the upper part of this stretch of river the deepening of the bed exceeded 3 m.; farther down stream, the bed was raised. Although this progressive change in the river bed dates back fully a century, it is by no means finished, and it is scarcely possible to predict when it will come to an end. However, in this case again, the cause is not to be traced back to levees.

During the writer's visit to the United States in 1927, he observed repeatedly that the Army engineers considered that there was no object in building a hydraulic laboratory. They contend that the accumulated results of experience gained by working with Nature itself are more valuable than the study of natural processes in the laboratory. They thus overlook the fact that most of the processes in the realm of hydraulics are of an extremely complicated nature,

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 961.

and that the observation of particular phenomena in Nature permits only in the rarest instances an analysis of the individual factors which have operated together to produce the phenomena observed. The investigation of cause and effect is the ultimate goal of science, and the hydraulic engineers in Germany are convinced, after 30 years' experience, that the hydraulic laboratory is an indispensable aid in the development of this science.

River problems are among the most difficult that the hydraulic laboratory is called upon to solve. The river in its natural state is the result of numerous factors, not only of a meteorological, but also of a geological, nature. The processes of flow, the distribution of the flow during the year—not only of the quantity of water corresponding to each stage, but also the duration of a particular flow, which fluctuates from year to year within certain limits—affects the course of the river. Only in the rarest cases is the problem confined to the same kind of geological formation in a river valley. It is quite possible to reproduce a stretch of river which has been accurately examined geologically and to investigate on this the effect of particular structures; but when the results of laboratory tests are to be carried over to Nature, one must always expect that, through accidents, through changes in the nature of the river bed, through snags or sunken vessels which are often invisible to the engineer, effects are produced which evade all attempts to reproduce them in a model test. Certainly, indications as to the best solution of a particular problem can be obtained through the laboratory, but there is little hope that, with the erection of one or more laboratories, the difficult problem of the Mississippi can be completely solved.

When, at the end of the Nineteenth Century, devastating floods occurred on various German rivers, there was appointed, at the command of the Kaiser, a commission whose function was to investigate the causes of the different flood catastrophes. The results of this investigation, which was conducted with unusual thoroughness, are compiled in a series of works which comprise an imposing library.

Mr. Freeman deals with the question of whether it is more practical to entrust the work to be carried out on the Mississippi to engineers in civil life in place of the existing military organization. This is a matter which must be settled within the United States itself.

The writer believes, however, that the author has made a very important suggestion, the application of which should not be disregarded. The Po, like every other river, is, in effect, a living organism, the study of which requires more than the lifetime of any single man. First, through systematic work, all the data can be brought together to give a unified picture of the organism which the engineer is to attempt to improve. Every interruption in the continuity of this scientific work is injurious. The unified and complete picture which Mr. Freeman has given makes it obvious that in more than a century of systematic work the fundamental facts for the investigation of all the factors affecting the formation of the river have been collected and assimilated.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

IMAGINATION IN CITY PLANNING

Discussion*

BY MESSRS. W. W. CROSBY AND ARTHUR A. SHURTLEFF.

W. W. CROSBY,† M. AM. SOC. C. E. (by letter).‡—Imagination—vision—and the necessity for either (or both) are admirably accented by Mr. Child. He suggests the need for their continuance and persistence with the growth of the community. Evidently Mr. Child regards the latter as a living thing, which fact seems sometimes disregarded by city planners.

In fact, the writer, after several years' experience with "city planning", has acquired the impression that only too often the "pictures" created have been of the "still life" order. They generally have been beautiful and symmetrical pictures, but not enough of them have conveyed any idea of "action". "Museum pieces" do not fill the needs of to-day alone on these lines.

Healthy, growing organizations depend for that life on their "circulations". In municipalities the circulation is the movement of the people, their supplies, their products, and their wastes. Unless this is properly provided for, so that it may be convenient, easy, free, speedy, and generally satisfactory, without danger either of "thromboses" or of arterial and even capillary ruptures—so to speak—under stress (peak loads), the right sort of imagination has not been used in making the "city plan".

It occurs to the writer that city planners can find stimuli to their imaginations and real aid to their art by studying some of the modern developments of physics, such as those set forth by Einstein, or in the later theories of atomic science, or even in the "cosmogony" of Jeans, Lodge, Millikan, *et al.*

The fundamental relation between circulation and regional planning is suggested by a statement, made elsewhere,§ that "intensive building cannot

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* This discussion (of the paper by Stephen Child, M. Am. Soc. C. E., published in April, 1928, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Coronado, Calif.

‡ Received by the Secretary, May 23, 1928.

§ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, pp. 1165-1167.

be carried out over large areas without causing an impossible traffic burden on street systems already fixed". * * * "Buildings of large bulk, therefore, must be balanced against excessively low buildings or open spaces". This is evidently based on considerations of circulation as well as the symmetry of the picture.

As an illustration of the need for imagination in planning consider the following. Congested traffic is now customarily controlled by the "stop and go" systems, familiar to all, in places where rotary or other and better automatic solutions of the needs have not been found practicable. Many city plans are being made on the basic conception that such "stop and go" control of traffic (including pedestrian) will prevail in these cities for an indefinite future. Is that assumption the proper resultant of a sufficient imagination, such as Mr. Child evidently believes necessary?

At most intersections now, where the "stop and go" control prevails even over pedestrians, it will be found that great inefficiency—which is rapidly getting worse—comes from the simple fact that the pedestrian streams are not checked at the building lines but are allowed to pile up at the curbs or gutters of the roadways.

Checking these streams at the building lines might be possible, but it would be difficult with these lines as they are customarily established; and the "control" of traffic, while at times necessary, is never so satisfactory as its automatic regulation of itself (such, for instance, as by rotary motion at the intersection of traffic streams). Surely engineering imagination should regard "control" as a "last resort".

A proper comprehension of this problem, with sufficient recognition of the mobility, that is, the "life" or "energy" involved with the "circulation", might, through better imagination, produce such an improvement in this detail of the city plan (intersections) as possibly in turn would result in greatly better city plans as a whole.

ARTHUR A. SHURTLEFF,* ESQ. (by letter).†—Imagination of the creative kind that Mr. Child describes is not of course a prerogative of any one profession. Even the most mechanical workmen are sometimes vitalized by good imagination. The man who invented logarithms had vast imagination. The individual who devised the slide-rule was an imaginative giant. It is pretty hard to say who lacks imagination. Certainly, many second-rate painters, poets, musicians, architects, and landscape architects lack this vitalizing strain in their fiber. For these men a lack of imagination means a pretty straight path (although sometimes too slow) to the poor-house and to oblivion. On the other hand, there have recently appeared some astonishing examples of imagination arising from unexpected sources—traders who have become great writers; elevator men who have become painters; and hack-humorists who have become leaders of intelligent public opinion.

The real test is of course the work which a man produces. If it is as dry as dust he is declared to be unimaginative. If he is effervescent, people say

* Landscape Archt., Boston, Mass.

† Received by the Secretary, May 23, 1928.

that imagination has ruined him. If he is a level-headed man and possesses the desirable quality of imagination, every one recognizes him as a genius whether he is in the profession of engineering, landscape architecture, or is a mere mathematical drudge like Mr. Einstein who suddenly saw the universe itself in a new light.

The need in city planning is, of course, the combined abilities of the best engineers, landscape architects, architects, and sculptors working in co-operation. For the best results, these men should possess the kind of imagination Mr. Child describes. If some of these men lack it, the other men may be able to leaven the lump sufficiently to save city populations from boredom, waste effort, and waste of money. If all these men lack creative imagination, then the populations are doomed unless by happy chance a kind natural topography or a lack of funds mercifully intervenes.

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PAPERS AND DISCUSSIONS

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LOAD DISTRIBUTION IN HIGH ARCH DAMS

Discussion*

By Messrs. William Cain and B. F. Jakobsen.

William Cain,† M. Am. Soc. C. E. (by letter).‡—The author has advanced the solution of the arch dam very materially by his analysis and the construction of the many diagrams as an aid in effecting the extended numerical computations required. The solution is a practical one for the dam of uniform thickness when the structure is not cracked or subjected anywhere to greater tension than the concrete can stand. However, if there is sufficient tension at certain sections to cause cracks for a suddenly applied load, it may be best to consider these cracks in order to be on the safe side; although for the very slow application of the water load, as the author contends,§ the "plastic flow" of concrete will modify results and perhaps at some sections, prevent "the tensile stresses from reaching the point indicated by theory". This is a consoling theory, but its realization was not much in evidence at the Stevenson Creek Experimental Dam where a large number of cracks formed. The theory must fit the facts.

As nearly all dams are not fixed at the base, in the sense of supplying sufficient tensile resistance there, cracks will occur at the base and a theory which ignores them cannot give correct results.

In the paper‖ by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., the parts of the sections of both arches and cantilevers in tension are neglected, and only the part of the section in compression is regarded as effective in computing moments. This, of course, is going to one extreme, for concrete can resist a tension of 100 to 200 lb. per sq. in., and the possible

* This discussion (of the paper by R. A. Sutherland, Assoc. M. Am. Soc. C. E., published in April, 1928, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

‡ Received by the Secretary, June 6, 1928.

§ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1039.

‖ "Analysis of Arch Dams by the Trial-Load Method," *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 61.

modification due to plastic flow is neglected, but the hypothesis is on the safe side. In fact, considering other influences that are usually neglected, such as shrinkage, variations, in *E*, water-soaking, and the elastic yielding of the foundation and abutments, it seems best to take the extreme view and consider as effective, for resistance to moments, only the part of a section in compression, particularly where the supposed crack covers a large part of the cross-section.

The author's analysis makes use of the ellipse of elasticity and thus abbreviates certain developments, and especial thanks are due him for the many labor-saving diagrams presented. For a dam that remains intact under the loads and temperature changes, the solution is complete, and even when cracks may be expected, the solution is valuable in a preliminary investigation. As the author affirms, the application of the method to many arch dams, leads to the conclusion that the normal load on a horizontal arch is not uniform.

One phase of the author's method is not clear. It is well known that there is one point on any assumed vertical beam (or cantilever),* considerably below the crest, where all the water load is carried by the horizontal arch and none by the vertical beam. In Fig. 5† or Fig. 18‡ no such point seems in evidence.

The writer has computed by the work method,§ the horizontal thrust, P_0, and the moment, M_0, at the crown for the circular arch of uniform thickness, t, and the central angle, $2\,\phi_0$. as well as the radial deflection anywhere for the two symmetrical radial loads, P, each making the angle, β, with the crown section,‖ but the resulting formula for deflection was found to be more cumbersome than the one derived by the author. The formula for the horizontal thrust, P_0, is,

$$P_0\,D_n = [(\phi_0 - \beta)\sin\beta - \sin\phi_0 \sin(\phi_0 - \beta)]\left(1 + \frac{k^2}{r_n{}^2}\right)$$

$$+ 2.88\,\frac{k^2}{r^2}\,[(\phi_0 - \beta)\sin\beta + \sin\phi_0 \sin(\phi_0 - \beta)]$$

$$+ 2\,\frac{\sin\phi_0}{\phi_0}\,[1 - \cos(\phi_0 - \beta)]$$

in which,

$$k^2 = \frac{1}{12}\,t^2;\; r_n = \text{radius of neutral axis};$$

$$\frac{k^2}{r_n{}^2} = \frac{1}{12}\left(\log_e \frac{r_e}{r_i}\right)^2;$$

r_e = radius of extrados;

r_i = radius of intrados; and,

* Point K, Fig. 15, in the writer's discussion of the paper entitled, "Gravity and Arch Action in Curved Dams", by Fred. A. Noetzli, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIX (1921), p. 74.

† *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1037.

‡ *Loc. cit.*, p. 1068.

§ The derivation being similar to that used in *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 529, 541.

‖ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, Fig. 15, p. 1062.

$$D_n = (\phi_0 + \tfrac{1}{2} \sin 2 \phi_0) \left(1 + \frac{k^2}{r_n{}^2}\right) - \frac{1 - \cos 2 \phi_0}{\phi_0} + 2.88 \frac{k^2}{r_n{}^2} (\phi_0 - \tfrac{1}{2} \sin 2 \phi_0)$$

The thrust, P_0, is assumed to be exerted at the center of the crown joint and the corresponding moment there, counter-clockwise moments being positive, is:

$$M_0 = P\, r_n \frac{1 - \cos (\phi_0 - \beta)}{\phi_0} - P_0\, r_n \left(1 - \frac{\sin \phi_0}{\phi_0}\right)$$

The author takes the horizontal thrust, H, as acting at the elastic center of the arch with a corresponding moment,

$$M = P\, r \frac{1 - \cos (\phi_0 - \beta)}{\phi_0}$$

This shows the relation between M and M_0. Presumably, when the mean radius, r, is substituted for r_n, P_0 should equal H.

As a check on the formulas for P_0 and M_0, $p_e\, r_e\, d\, \beta$ (p_e being the unit load on the extrados), was substituted for P and the result integrated between the limits, $\beta = 0$ and $\beta = \phi_0$, to give P_0 and M_0 for the full normal load, and the results were found to agree exactly with previously derived formulas.*

In the formula for P_0, the term having the factor, 2.88 shows the influence of shear; and since there is some uncertainty as to its numerical value the figure, 3, can be substituted for 2.88, to agree with the author's assumption. The hope of the designer is that some method of shortening the work materially by means of curves will be devised. In that direction, the author has gone a long way, and he will receive the thanks of the profession.

B. F. Jakobsen,† M. Am. Soc. C. E. (by letter).‡—In Appendix I§ the author gives the derivation of formulas for stresses, adapted from Professor Guidi. These formulas apply for thin arches only, because of the assumption that the elastic center of gravity coincides with the center of gravity of the geometrical axis.‖

The distance, $O\,G$, is the writer's A', and this is:¶

$$A' = r_n \frac{\sin \phi_0}{\phi_0} \qquad (89)$$

in which, r_n is the radius to the neutral axis and ϕ_o is one-half the central angle. When r_m is the mean radius, or the radius of the geometrical center line, then:**

$$r_n = r_m - c \qquad (90)$$

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), Equations (103) and (105), p. 531.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, June 30, 1928.

§ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1044.

‖ *Loc. cit.*, pp. 1046–1047, Fig. 7 and Equation (15).

¶ "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), Equation (18), p. 484; see, also, Professor Cain's discussion, Equation (105), p. 531.

** *Loc. cit.*, Equation (21), p. 485.

and,

$$c = \frac{k}{1 + k} r_m$$

and for a rectangular section,

$$k = -1 + \frac{r_m}{t} \log_e \left(\frac{r_e}{r_i}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots(91)$$

in which, r_e and r_i are the radii of the extrados and intrados, respectively. The assumption, made by the author, that $c = 0$, is true, or approximately true, only for thin arches, as was shown by the writer and also by William Cain, M. Am. Soc. C. E.* The author also assumes that the trapezium law† is correct; this also is approximately true only for thin arches,‡ as previously shown by the writer.

The derivation of stresses, as given by the author in Appendix I, is based on the same assumptions as were made by Professor Cain,§ and the results, therefore, should be identical. As a check, assume an arch for which $\frac{t}{r} = 0.2$ and one-half the central angle, $\phi_0, = 45$ degrees. From Equation (22)‖ is found, $K_0 = 1.0267$ and from Equation (34),¶ $f_1' = -11.98\ p$, whereas Professor Cain's formulas give $12.295\ p$, a difference of less than 3%, which probably is due to inaccuracies in the calculations. To the writer's way of thinking, Professor Cain's derivation is simpler and clearer, and the same holds for the writer's own derivation, as given in the paper referred to. That may be only a personal preference and is at any rate immaterial, the essential point being that the author's equations are true only for thin arches, such as $\frac{t}{r}$ less than 0.3. The same remark applies to his calculation of stresses due to temperature variations and to his calculations for deflections.

In investigating the deflection produced by non-uniform water pressure,** he assumes that the load is symmetrical and that the two semi-arches are symmetrical also. The assumption that the load is constant for any arch is undoubtedly always incorrect, and so is the assumption of symmetry.†† The writer's experience with the design of arch dams leads him to believe that it may not be worth while to attempt to take account of the non-uniform load on the arch, unless the question of symmetry is likewise taken into consideration. After all, the main purpose is to design a safe dam and, therefore, to make such assumptions as will lead to a stress determination which is first of all safe and, in addition, a fairly close approximation to the actual stresses. It is in this respect that the cylinder formula is lacking, because the stresses found are too small and sometimes much too small.

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), Equations (18) and (105), pp. 484 and 531.

† *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1048, Equation (25).

‡ *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), Fig. 52, p. 581.

§ "The Circular Arch Under Normal Loads," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

‖ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1048.

¶ *Loc. cit.*, p. 1049.

** *Loc. cit.*, p. 1058.

†† See, for example, Figs. 1(*a*) and 1(*c*), as given by the author, *loc. cit.*, p. 1031.

The author seems to have assumed throughout his paper that the arch load at the abutment is zero.* This assumption is not correct, as the writer has pointed out in discussing another paper.† The author also assumes that the foundation is practically unyielding.‡ The writer raised this question some years ago§ and Dr. Fredrik Vogt has dealt with it recently and has shown that both the cantilever deflections and the arch deflections are materially increased by the yielding of the foundation‖ and that the division of load between the cantilevers and the arches is influenced by the yielding of the foundation to a considerable degree. The measurements on the Stevenson Creek Test Dam show yielding of the foundation, which checks closely with the formulas of Dr. Vogt.¶ The author also neglects shrinkage, but as the writer has recently dealt with this quite extensively,** there seems to be no necessity for repeating it. The report on the Stevenson Creek Test Dam also shows that shrinkage produced a very considerable cracking of the dam, and it is not clear to the writer that this can be neglected as being inconsiderable in calculating the stresses.

The author suggests that arch dam design may be improved by varying the radius of the horizontal arches, in order to adapt them better to the non-uniform arch load. Stucky†† assumes that the center line of the arch coincides with the funicular polygon and that, therefore, generally the center line is not an arc of a circle. He does not, however, investigate the problem, but states that in most cases the center line will vary so little from a true circle, that it may be assumed to be circular. Some years ago, after reading the paper by C. S. Whitney, M. Am. Soc. C. E., entitled "Design of Symmetrical Concrete Arches",‡‡ the writer made several attempts to determine the most economical shape of the center line. He came to the conclusion that due to the many uncertainties such as shrinkage, temperature variations, yielding of the foundation, etc., the problem is too complicated and moreover that the shape of the most economical center line will vary as the volume of concrete, due to temperature variations, shrinkage, time effect, etc., so that little if anything can be accomplished with any degree of certainty.

The possibility of improving arch dams lies in the direction of a rational grouting of the contraction joints. In this way the dam may be made into a monolithic structure, which will have compressive arch stresses in places when the reservoir is empty; but it is not of vital importance to know whether the stress is 500 lb. per sq. in., or 10% more or less. It is important, however, to know that tension does not exist, since concrete cannot be depended on to resist tension. Dr. Vogt has introduced a system of grouting, which permits of varying the grouting pressure so that, for instance, a low grouting

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, Figs. 5(*d*) and 5(*j*), p. 1037, and Fig. 16, p. 1063.

† *Loc. cit.*, p. 1311.

‡ *Loc. cit.*, p. 1034.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 102.

‖ *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 554.

¶ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, Report on Arch Dam Investigation, Vol. 1, p. 127, and Figs. 81 and 82, p. 128.

** *Loc. cit.*, Papers and Discussions, p. 1479.

†† "Etude sur les barrages arques," Lausanne, 1922, "Calcul des arcs," p. 13.

‡‡ *Transactions*, Am. Soc. C. E., Vol. 88 (1925), Article IV, p. 966.

pressure is applied at the intrados at the abutment, where the compression would otherwise be high, and a high grouting pressure can be applied at the extrados of the abutment where otherwise a considerable tension may develop. It seems to the writer that such a system of grouting would be of considerable value in perfecting rational design and in the construction of arch dams.

The author correctly states that the cantilevers must be considered as having vertical radial limiting planes and not vertical parallel planes, as is generally assumed. This is of especial importance in gravity dam design, as the writer has recently shown.*

The author's Fig. 2† should be of assistance in determining the division of the water load between cantilevers and arches. If an approximation can be obtained on which to base the first set of calculations, these may be materially shortened, and even if absolute agreement is not reached, the result may be close enough for a stress determination.

* "Notes on Arched Gravity Dams", *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1135.

† *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1033.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

EVALUATION OF WATER RIGHTS

Discussion*

By Messrs. Joseph Jacobs, Fred H. Tibbetts, Charles R. Hedke, Fred C. Scobey, Lyman E. Bishop, and R. L. Parshall.

Joseph Jacobs,† M. Am. Soc. C. E. (by letter).‡—This paper is of value and interest because it deals with a subject that grows in importance each year, in accordance with a steadily expanding population, agriculture, and manufacturing industry and their persistent demands for additional water supplies. It seems to deal, not with the broad, general subject of water-right valuation, as its title implies, but rather with a restricted phase of it which has to do more particularly with the relative values of several irrigation water rights in the same mutual ditch system, but under decreed differences as to priorities and quantities of water.

It is frequently contended, in the valuation of public utility properties, that no allowance should be made for water rights, because the actual ownership of the water rests with the public, and there have been many findings and rulings by public service commissions sustaining this view. As against allowing nothing for the water right, a more equitable showing could be made for the principle of dividing the water-right value between the public and the utility owner, in cases where the water-right value is based on consideration of the extra cost involved in developing the next best available water supply. This would be based on the theory that the consumer public would otherwise lose, completely, the money value of its natural advantage of location with respect to the cheaper supply. That water rights do have value, however, and that this shall be recognized as a principle in public utility valuations, unless there is definite stipulation to the contrary, has been too often attested in decisions of the higher Courts to be seriously questioned. In the recent

* This discussion (of the paper by John E. Field, M. Am. Soc. C. E., presented at the meeting of the Irrigation Division, Denver, Colo., July 14, 1927, and published in April, 1928, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. (Jacobs & Ober), Seattle, Wash.

‡ Received by the Secretary, September 2, 1927.

Indianapolis Water Case the United States Supreme Court stated that "the water rights must be included".

Further evidence that owners attach value to their water rights inheres in the history of the violent and costly struggles and encounters, both physical and judicial, that have been engaged in to establish and maintain such rights.

The Federal Power Commission also, in a way, recognizes this principle of water-right value. In its published rules and regulations which provide (in respect to power developments on Government land within its jurisdiction) that the licensee shall pay an annual fee, based on the installed capacity of the plant up to the power capacity of the site, ranging from 5 cents per h. p. for the first year of operation to 25 cents per h. p. for the third and subsequent years of operation. These figures have definite, capitalized values depending on the assumed earning capacity of money and, with a known power head for any given case, these values could be expressed in terms of dollars per second-foot of water. While it is believed that thus basing the charge on plant capacity may indicate recognition, by the Commission, of the principle of water-right value, the charge is not, of course, a measure of such value. In the first place, the Commission states that the charge is "for reimbursing the United States for the cost of the administration of the act." In the second place, the charge may be considered as being in part for rental of land occupied by the project and, if the available water supply is in excess of any possible power-market demand, it may be regarded as attaching more to the power franchise than to the water right, because such a distinction can be made.

It is less difficult to prove that water rights have a value that should be recognized, than it is to determine what that value is. The valuation method to be used, and the final value to be determined, depend in part on the purpose of the valuation; in part on whether a private industry or a public utility is being dealt with; and, finally, on the elements that make up the value of the water right. It is not possible to formulate a simple definition of what these elements are because of the many qualifications and exceptions that have application to specific cases. Moreover, by reason of statutory differences, what may be valid in one State, respecting water rights and their valuation, may not hold in another State. There are numerous factors that have a bearing on water-right valuation. Some of the more important of these are:

(*a*) The State laws and regulations on which the validity of the water right rests.

(*b*) The priority, amount, and duration of the water right as defined by Court decree or as affected by franchise provision. This involves also the question of storage rights as distinguished from normal-flowage rights.

(*c*) The franchise, and the limitations thereof, under which is operated the project to which the water right attaches. A short-lived franchise would detract from the value of a water right and the franchise may even stipulate that no value shall attach to the water right under certain conditions.

(*d*) The amount that was legitimately expended, by purchase or otherwise, in securing the water rights.

(*e*) The value, for agriculture or other ordinary purposes, of the land occupied in the development of the water right as, for instance, the land required for dam, power house, canal head-works, etc.

(*f*) Whether or not the project to which the water right attaches is fully or only partly developed and whether the market for the product that the project supplies is immediate or only prospective and whether it is permanent.

(*g*) The abundance of the water supply. A supply in excess of present and prospective market requirements would tend to depreciate, as would a restricted supply tend to appreciate, the unit value of a water right.

(*h*) The cost of the next best available water supply or other adequate substitute.

(*i*) A possible higher valued alternative use for the water.

(*j*) The capitalized value of net earnings of the project above operating expense and normal interest return on the investment in the physical plant.

The bearing of most of these considerations on the value of a water right is too obvious to require comment, but a brief further discussion of some of them may be desirable. Referring to Item (*d*), it would seem that while the amount that was legitimately expended in securing the water right would not necessarily fix its maximum value, it would normally fix the minimum value that should be allowed. The word, "normally," is used because if the conditions surrounding the business were such that it was impossible to earn a fair return, regardless of rates, and such instances can be cited, then the water right might have no value at all. The Item (*e*) consideration affords another criterion of value. If the value, for ordinary purposes, of the land required to utilize the water right, is deducted from the actual price paid for the land, or if it is deducted from the value of the land for ordinary purposes multiplied by the average public utility factor that obtains in the district, in connection with real estate purchases, then the remainder may be regarded as one measure of the value of the water right. This, however, is not deemed to be a rational or dependable method of water-right valuation because of the uncertainties and the fortuitous circumstances that affect the "public utility factor". In many instances the land has no value at all for ordinary purposes.

In respect to Item (*f*) it may be said that the status of the project, as to the extent of its development and the character of its market, must be reflected in the value of the water right, although it is believed that these factors would be more fully and more properly reflected in "Development Cost" and "Going Concern Value", if the intangible elements of value in a property were segregated instead of being combined into a single item, as is so frequently done. While a developed project with good markets for its product would tend to give substantial value to a water right, no Court would be inclined to concede a material present value to such right on account of a prospective future market unless that market was definitely assured and reasonably close in point of time.

In connection with Item (*g*) which indicates that the magnitude of the water supply, in relation to demand, affects the water-right value, a distinction between intrinsic value and commercial or market value must be recog-

nized. In the semi-arid States, for instance, irrigation water has intrinsic value because agriculture would be impossible without it, but, even if the results of this agriculture are highly profitable, it does not necessarily signify that the water right is of high commercial value. If the available water supply is in excess of possible demands, and is obtainable from the State without cost other than nominal filing fees, then, manifestly, the water right proper could have no great commercial value, for no one would pay a high price for that which he can obtain practically for nothing. Such value as was warranted by the net profits from the agriculture served, would be reflected mostly in the land and, according to the extent that it was reflected in the stock of the ditch company, the value of the latter, in excess of that represented by physical plant investment and development costs, would more properly attach to "Going Concern Value" and to "Franchise Value" than to water-right value. If, however, there was not an abundance of water for future or other requirements then the water right would properly absorb a greater part of the excess value previously mentioned.

The cost of the next best available water supply or other adequate substitute and the possible higher valued alternative use of the water, referred to in Items (*h*) and (*i*), not only afford distinct bases for determining the value of a water right, but they may, in certain cases, become the controlling considerations. Certainly, they would have application in dealing with water supplies for private industries and, in connection with public utility properties, they would, ordinarily at least, fix the maximum allowance that could be made for the water-right value. If the water right may appreciate in value, or attain maximum value, from these considerations, a logical corollary would be that if in the future, through changed conditions in stream run-off, or through new scientific developments, a substitute may be provided at lesser cost, then the value of the old plant would depreciate and this loss in value would also, in part, be reflected in the water right.

The most fundamental consideration, the one that has the most general application, is that defined in Item (*j*). The capitalization of the excess net earnings mentioned, measures not the value of the water right alone, but of all those elements classed as "intangibles". If there is no such excess, actual or prospective, then, from an investment standpoint, no value could be assigned to the water rights or other intangibles, and the allowed value for a property would need to be assigned wholly to its tangible elements. Intangibles are frequently lumped in a single item because that may suffice for the purpose in hand, but for any refined valuation they should be segregated, and particularly so, when dealing with the specific inquiry of the value of a water right or any other single intangible. This lumping of intangibles tends to confusion, and not infrequently this lump sum is expressed wholly as a water-right value when that may not be the case at all. The intangibles may include "development costs", "going concern value", franchise value, water-right value, etc. The first two are things apart and rather definitely determinable. Franchise value and water-right value, however, may sometimes be confused, but each individual valuation case will usually afford some rational basis for segregation.

As applied to private industry, the logic of capitalizing excess net earnings as a basis for determining water-right values can hardly be questioned. It may also be correct as applied to a public utility, depending on the terms of the franchise under which the utility is operating, and, particularly, if the valuation case is one in which the rates and earnings of the property are found to be fairly definitely established, regardless of the method by which those rates may have been reached. This, in a sense, begs the question, in respect to those public utility properties for which, in theory at least, the rates depend on the rate base rather than the reverse, and for rate-making purposes in such cases a regulatory commission would need to use some independent method of arriving at the water-right value.

All these considerations and methods of determining water-right values may have application in certain cases yet none of them has universal application to the exclusion of the others. They may lead to quite different results and the one that should control depends on local conditions and on all the circumstances surrounding the particular case in hand. There is too much tendency on the part of appraisal engineers to adhere to rigid values in valuation work, some adopting that method which always yields a high valuation and others that which always makes for a low valuation. There are, of course, certain fundamental principles that must be observed, but an important principle that deserves broader recognition than it usually receives, one that will tend to dispel the prejudicial view, is this: That no single method of valuation can be rigidly applied to all cases, that each case is a thing apart, and that the valuation method, to be applied, must depend on, and be adapted to, all the circumstances and conditions surrounding the particular case and business involved. Valuation work is complex at best, and to arrive at just results the appraisal engineer must possess, in addition to his technical knowledge, a discriminating fairness and an abundance of good judgment and common sense.

The author states much that the writer fully concurs in and there are only a few points to which he thinks some exception might be taken. To these latter he desires chiefly to direct his comment. The author states that in building new irrigation works to replace old ones, the amount paid for those parts of the old works that were abandoned or destroyed, was, of necessity, added to the cost of the water or the water right. This, it seems, is properly a development cost and in no sense a proper charge against the water right. That it would add no value to the water right proper is evidenced by the fact that no one would pay this extra cost for the water right alone if, for instance, there was an abundance of water in the stream. This development cost would be reflected, of course, in the price placed on the stock of the company, but it would not, in the writer's judgment, represent water-right value.

The author also states* that in his early appraisals it was deemed merely necessary to ascertain the prevailing sale value of water. When conditions are stable the market or sale price of water stock is at least a criterion of value that should be considered and in some cases it may be important. There

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1071.

are cases, however, where the market price of water stock is largely fictitious, bearing but little relationship to earning capacity or to actual investment. In such cases it would not be safe to deduce a water-right value from the market price of the water stock. The water-right value must, of course, be segregated from the all-inclusive stock value. At another point, the author himself recognizes the unreliability of this market price method of determining value.

The author's statement* that stored water has little more value than unstored water requires some qualification. The truth of it depends much upon the flow characteristics of the stream involved. If the unstored or normal flowage water is exhausted during the very early months of the irrigation season, as it frequently is, then, even for identical volumes, the stored water would be more valuable to the crop than the normal flowage. In general, it is believed that stored water is more valuable, volume for volume, than unstored water, because it is under definite control, it can be delivered as, and when, required, and it is available in periods of the most acute need. The comparative values of like volumes of stored and unstored water, however, may be something quite different from the comparative values of a water right from normal flowage during the irrigation season and a water right for storage during the non-irrigation season. The exact definition of the water right and the stream characteristics are all important, because the average number of acre-feet of usable water that each water right will produce annually must be determined before a comparison of values can be made. The term "usable water" is emphasized because, not only does the right usually cease when beneficial use ceases, but a water right takes on value only as it has a dependable water supply, the equipment to deliver the water, and a market to absorb it. In water-power development, and also in domestic water supplies, the rate of stream discharge rather than the total volume of run-off may often be the more important consideration, so here, again, its adaptability to such a condition, makes stored water more valuable than unstored water.

Where there are several water rights on the same stream the priorities of those rights, and their time limitations within the year, if any, are important elements affecting value, and the length of the stream-flow record may also appreciably affect the water-right value. The writer agrees with the author that a longer record than eight years is desirable. A short-time record cannot develop true stream characteristics and a long-time record is likely to reveal worse low-water conditions than the shorter record would show. The effect of this would be to give greater relative value to the early priorities, and it may also give a greater value per unit volume of water. The length and dependability of the available discharge records are elements that should not be disregarded in water-right valuation work.

The author, in his development of water-right values for several specific cases of canal systems,† does not indicate how the gross valuation of these properties was initially determined. He starts with an assumed, definite gross value, which is apparently the market value, and an assumed value for the

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1072.

† *Loc. cit.*, p. 1074.

physical elements of the property. Calling the difference between the gross value and the structural value, the value of the water rights, he develops a method of apportioning this latter value among the several water rights according to their priorities. The writer would first indicate that, since the water-right value results directly from the relative gross and structural values, the important primary consideration is the correctness of the method by which those latter values were determined. The author does not indicate what these methods were. This difference between gross and structural values is not necessarily the water-right value alone. It is the aggregate value of all the intangibles, and these may include elements other than water rights.

In the Fort Lyon Canal case* it is stated that the gross value of the system is $6 000 000, the structural value, $2 000 000, and that the water-right value is, therefore, $4 000 000 which, with an indebtedness of $600 000, gives a total water-right value of $4 600 000. It is not clear why the $600 000 should be added to the water-right value. If it represents value at all it is presumably included in the gross value of $6 000 000, and if it represents accrued operating losses that are in fact "development costs" they are not properly a water-right increment of value.

The author's method of segregating the total water-right value among the several rights† on the basis of their priorities is thought to be the logical and probably the only rational method for such segregation. It is believed, however, that in determining the quantity of water deliverable during the year under any decree, the possible yield for the full year should not be taken, but only the yield of that portion of the year during which the water can be put to beneficial use; as, for instance, during the irrigation season only. Unless the decree specifically permits it, and there are facilities to deliver, and a market to absorb the water deliverable during the non-irrigation season, the normal-flow water right for that period would have no value. Even storage rights for that season might have no value if the cost of the storage works would be greater than the value of the water stored. A recasting of the author's figures, on the basis of water yield during the irrigation season, will give an entirely different set of water-right values.

Determining the so-called efficiencies of the various decrees as bases for determining the value of a water right of 100% efficiency would seem to be an unnecessary elaboration because, after the value of the acre-foot of water is determined, one need only multiply that figure by 724 to obtain the theoretical value of a continuous flow of 1 sec-ft. The word, theoretical, is used advisedly because no such 100% efficiency can obtain, regardless of the provisions of the decree, unless the water is continuously available and there exists facilities for its delivery and a market to absorb it in beneficial use. It is possible that, with storage and the development of markets, these conditions may sometimes be met, but without these essentials satisfied, such figures as the author develops for the value of water rights of 100% efficiency are academic and likely to be misleading.

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1075.
† *Loc. cit.*, p. 1076.

FRED H. TIBBETTS,* M. AM. SOC. C. E. (by letter).†—To the present the engineer attempting to make a rational valuation of a water right finds little precedent and practically no authoritative literature to guide him. Mr. Field, accordingly, deserves all the abundant credit due for his important pioneer effort. The paper is of the greatest interest and value, containing a clear, concise, and logical method of fixing the valuation of a water right where the data required can be readily, and with certainty, procured.

At a time like the present, when agricultural securities are notably below par, and particularly in California, where State irrigation districts have issued more than $100 000 000 worth of bonds, it would be of material assistance in financing additional developments if the general public, bankers, and Courts, could be educated to a proper recognition of the true value of reliable water rights available with certainty for irrigation development. It is the writer's hope that this paper will assist materially in creating recognized methods of fixing substantial values for water rights.

Mr. Field's method is to obtain the total value of a public utility, or mutual water company, furnishing water for irrigation, by determining the market price of its total outstanding stock and presumably other securities. From the value thus obtained he deducts the appraised value of the physical properties and assigns the remaining value, if any, to the water right. The few public utilities serving water in California, seldom have a total stock value equal to that of their appraised physical properties. Hence, although it is recognized that their water rights have substantial values, Mr. Field's method of obtaining such values is quite inapplicable. As a broad generalization, it seems highly conjectural to assume that all the stock value in excess of the reproduction costs (less depreciation) of physical properties, represents the value of the water right. This residual must also include the sum of all "intangible values", such as promotion and development expenses, organization, "going concern", etc. The total stock value of any corporation is also measurably dependent on public confidence in the officers of the company, and many times on the efficiency of its advertising. There are many corporations in which at times the total market value of the stock may bear but little relationship to the value of the physical properties. A good example, in California, at present is a certain boom oil company luridly advertised, and overcapitalized, in which over-issued stock with no valuation whatever of physical properties back of it, may exceed $50 000 000 to $100 000 000.

The writer is somewhat of the opinion that Mr. Field's method attacks the problem from the wrong end. He is inclined to the belief that if he were to purchase a large amount of stock in the water companies which Mr. Field discusses, he would first appraise the physical properties and then, from independent considerations, would appraise the water rights, and, finally, from the total of these would determine what he considered to be the value of the stock. This procedure seems more logical than to accept public opinion regarding the market value of the stock as conclusive, and from this to assert that the dif-

* Civ. Engr., San Francisco, Calif.

† Received by the Secretary, September 2, 1927.

ference between this market value and the value of the physical properties must represent the value of the water rights.

The writer is in fundamental disagreement with the author when he concludes that the value of an acre-foot of water is the same, whether it is used for domestic purposes, or for those of manufacturing and agriculture. It is certain that in California the cities can and do pay far more for domestic water than the farmer can pay for irrigation water. Mr. Field's statement* that "except for the need of water for agriculture in the semi-arid region, there would be enough for domestic needs and for manufacturing", seems quite inapplicable to regions like Southern California, for example, where the duty of water per acre for urban development is about the same as for agricultural development, and where large quantities of water must be imported from far-distant points to satisfy the domestic and industrial needs alone. The writer sees no reason why the value of water for domestic purposes should not be based on the city's necessities and on what the city can afford to pay; in particular, he sees no connection between the value of water for domestic and for agricultural purposes, and can discover no reason why the value for domestic purposes should be based on the value for agricultural purposes.

Mr. Field suggests that the idea of converting the cubic feet per second, into acre-feet and basing the value of a water right on the number of acre-feet thus produced, at one time "appeared ridiculous". The writer is entirely in agreement with Mr. Field, however, in that this is probably the correct method. As a matter of fact, this is the only method for valuating a water right that the writer has ever used or considered. In determining the value of water rights for irrigation purposes the "useful yield" during the irrigation season should be determined in acre-feet, and the valuation based on this unit. The "efficiency" of a water right for irrigation purposes should then be the ratio between the number of useful acre-feet produced by this water right during the irrigation season and the number of acre-feet during the irrigation season (included in the irrigation project's demand curve) which would have been produced by a second-foot flowing continuously during the peak demand of the irrigation project, and at other portions of the season reduced in proportion to the project's reduced demand.

While the author's method is logical and excellent for the conditions outlined, the writer believes that recent appraisals of water rights on the San Joaquin River, in California, were based on broader and more general principles.

The problem presented was unusually complex and complete, because nearly all conceivable elements of value were present in substantial quantity. The important water rights on a major stream controlled by a single private interest were proposed for transfer outright to a newly organized water storage district. As it was conceded that the district would not attempt to condemn the water rights, the problem was not complicated by intricate and archaic legal theory, as is so frequently the case. The appraisals described herein led to a final valuation of about $9 500 000. This was a compromise figure

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1072.

agreed on by the writer, representing the owners of the water rights, and A. Kempkey, M. Am. Soc. C. E., Consulting Engineer for the District. The theories herein described are those developed jointly by Mr. Kempkey and the writer.

Chief consideration was given to the following elements:

1.—The market value as established or indicated by actual transfers of comparable rights under conditions similar to the one considered.

2.—The substitutional service value, or the cost of securing by some other method the same water supply as is obtainable by the appraised water right. Two other methods were readily available in whole or in part: (*a*) Impounding flood water; and (*b*) pumping ground-water.

3.—The water value, or the capitalized sales (per acre-foot), outside the District, of irrigation water at diversion points, under similar conditions.

4.—The land increment value, or, the net increase in value of the land to be irrigated, caused by the acquisition of a water supply for irrigation.

5.—The land decrement value, or the decrease in value of lands irrigated or partly irrigated under the appraised water right, arising from depriving them of their water right. This is an unusual condition, but it obtained in the proposed transfer, thus rounding out the problem. Water was to be taken from 178 000 acres of "grass lands" of inferior quality, irrigated for pasture, and transferred to new lands as yet unirrigated and arid, but of superior quality.

1.—Market Value.—It appears to be the general legal opinion that "market value" as shown by the sales of other water rights of a similar character under essentially analogous conditions, constitutes the true criterion for a valuation of a water right. Like most legal formulas, this opinion is theoretical and evasive. The "market value" of wheat may be thus determined. Water rights cannot be standardized; no two are alike; there are few *bona fide* sales in which the price paid for the water right can be clearly identified; and in the last analysis final conclusions are subject entirely to matters of individual, professional, and expert opinion, as to the degree of comparability of the water right appraised with other water rights the market value of which has been estimated from purported sales.

It would also seem obvious that if there were sufficient sales to establish a "market" for water rights, then the value thus established must take into consideration all the usual elements of supply and demand in ordinary commercial transactions. In particular, it must be chiefly affected by the value of the water right to the purchaser, and its value to the seller. These two values must, in time, reflect chiefly the values determined by Elements 2 to 4, and particularly the last two, Elements 3 and 4.

Included in voluminous expert testimony in the Spring Valley Water Company rate cases in San Francisco, Calif., are deductions for the market value of irrigation water obtained by this method and varying from $70 200 to $1 272 per sec-ft. It will be obvious that the results obtained are entirely a matter of professional judgment as to the degree of comparability of the different water rights.

2.—Substitutional Service Value.—It is not always possible to obtain any substitutional service value because frequently no source of water supply can be obtained other than the one appraised. The writer believes, however, that much weight should be given to this matter whenever it can be clearly shown that there are one or more possible sources of water supply other than the one appraised. In many cases reservoir construction for the storage of flood water will furnish an entirely satisfactory, substitutional source of supply. Opinions may differ as to the relative value per acre-foot of irrigation water thus obtained. The writer believes that irrigation water obtained from a natural stream flow, the yield of which is known from long and reliable records, can be more accurately estimated, and hence, is more valuable, than a corresponding quantity of water which is merely the estimated yield from a storage reservoir not yet built. Mr. Kempkey, however, held that reservoir releases per acre-foot are more valuable than corresponding natural stream flow, because the construction of the reservoir allows, in the future, much greater elasticity of operation and variation in reservoir releases with variable seasonal weather, and variable crop production, over a long period of years.

Substitutional service in large part may also be obtained from wells, but here, again, the cost of obtaining a well supply, as yet unconstructed, cannot be estimated over a long period of years with any great confidence, because of the probable variation in pumping head and in cost of power.

3.—Water Value.—It is generally much easier to obtain reliable records of the sale of natural stream flow for irrigation ditch diversion, or of reservoir releases for irrigation purposes, than similar values of the sale of water rights. However, it may still be a matter of opinion as to the degree of comparability of the water sold, with that to be furnished under the right appraised. When reliable figures for the market value per acre-foot of irrigation water are thus obtained, they can be capitalized and used as a measure of the value of similar quantities of water to be obtained under the rights appraised.

4.—Land Increment Value.—Increase in the net value of irrigated over unirrigated land can usually be determined with considerable confidence, providing irrigation has been long practiced in the district under consideration. In obtaining the net increase in value of irrigated lands because of the irrigation water rights, the difference in value between the irrigated and the adjacent unirrigated land without water rights should be obtained, and from the difference thus obtained, must be taken the cost of improving the land, leveling it, checking it, constructing irrigation distribution, etc. Not all the net increase in value, as thus obtained, can be claimed for the water right; a portion should be claimed by the land and a portion by the agency which has provided and operated the physical works for the diversion and conveyance of the water.

In this connection it may be emphasized that at the instant the physical works for diversion and conveyance have been completed and the land owner has prepared his land for irrigation (with no water having been actually diverted), there does not exist a completed and valuable right in the sense that such a right will exist after many years of diversion and use. It may be assumed, therefore, with propriety that the water agency on the one hand

and the consumer on the other, after years of operation, have contributed jointly (through diversion in one case and use in the other) to the creation of a property right having a very substantial value, to wit, a perfected water right. It will also appear that the contribution in each case has been independent of investment, and is based solely on the physical fact that the water agency, on the one hand, has diverted the water, and the consumer, on the other hand, has used it. Obviously, diversion by the water agency would be of no value without use by the consumer and in the case of the particular land in question, use could not have been effected without diversion by the public utility. It appeared, therefore, equitable that each should share equally in the property value thus jointly created.

5.—Land Decrement Value.—This method is clearly inapplicable, except as to such lands as propose to give up their water right, resulting in their being dried up and rendered less productive. In the case considered, the value to the owner (of the dried-up lands and of the water right) was obtained by estimating the difference in value of the land, with and without the irrigation to which it had been accustomed, or by capitalizing the difference in revenue or rental from the same lands, with and without their accustomed water supply.

Appraisal Units.—To obtain a quantitative expression of the valuation of the different water rights the "safe net yield", in acre-feet, during the assumed irrigation season, was determined for each water right. The "safe net yield" was a term applied to the estimated minimum quantity of water allowable in the minimum or dryest year. The normal water supply at the diversion points in the entire district of 554 300 acres was estimated at 2.45 acre-ft. per annum, with a monthly distribution as shown by Fig. 2 and Table 1. It was assumed that shortages in minimum years were allowable up to $33\frac{1}{3}\%$ of the normal supply. Fig. 2 shows the assumed water demand of the district.

TABLE 1.—AVERAGE ACTUAL YIELD OF THE WATER RIGHT.

Month.		January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.
District demand	Percentage per month..	0	0	6	12	21	20	17	13	9	2	0	0	100
	1 000 acre-ft. per month.	0	0	23.3	46.6	81.6	77.8	66.1	50.5	35.0	7.8	0	0	388.7
	Acre-feet per month per second-foot...........	0	0	17.1	34.3	60.0	57.1	48.5	37.1	25.7	5.7	0	0	285.5
Actual total..	1 000 acre ft. per month.	14.9	15.0	29.0	50.1	77.8	79.0	77.0	49.5	27.8	22.5	16.5	15.6	474.7
	Acre-feet per month per second-foot...........	11.0	11.0	21.3	36.8	57.1	58.0	56.6	36.4	20.4	16.5	12.1	11.5	348.7
Yield useful..	1 000 acre-ft. per month.	0	0	23.3	46.6	77.8	77.8	66.1	49.5	27.8	7.8	0	0	376.7
	Acre-feet per month per second-foot...........	0	0	17.1	34.3	57.1	57.1	48.5	36.4	20.4	5.7	0	0	276.6

A hydrograph, based on long years of record of the actual yield of the principal water right appraised, is superimposed. Under these conditions a theoretically perfect water right, that is perfect in monthly distribution, would yield 285.5 acre-ft. of useful water per sec-ft. during the irrigation season.

The relative values of different water rights appraised were determined by superimposing on the hydrograph of the water right, as obtained by measurements over a period of years, the irrigation demand curve of the district, and their relative value was determined to be strictly in proportion to the number of acre-feet which each right would produce within the demand curve of the district.

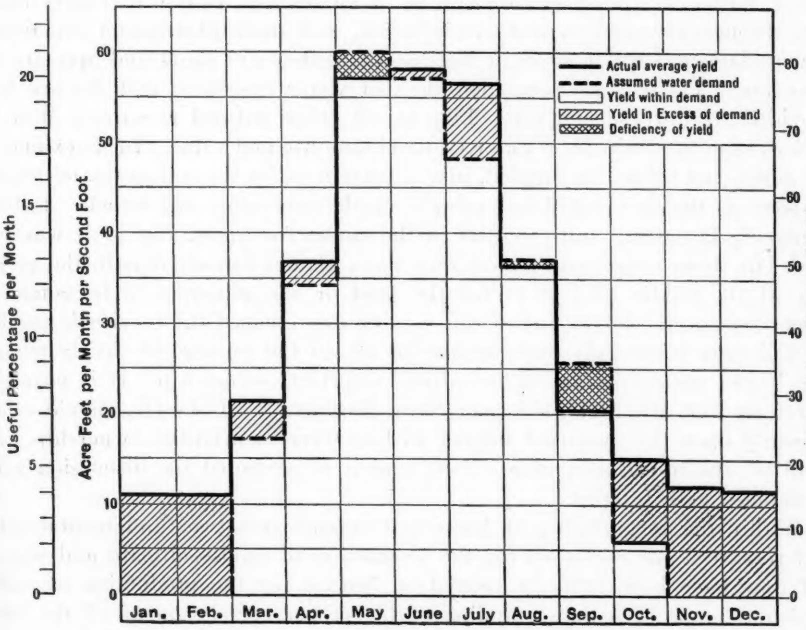

FIG. 2.—AVERAGE ACTUAL YIELD OF THE WATER RIGHT.

The final conclusion was that water rights from the San Joaquin River, for acquisition by the San Joaquin River Water Storage District, delivered at the head of the distribution canals, were worth $30.40 per acre to be irrigated with a normal demand of 2.45 acre-ft. per acre, or were worth $18.58 per acre-ft. of "safe net yield" obtainable during the minimum year of record.

CHARLES R. HEDKE,* M. AM. SOC. C. E. (by letter).†—In the early stages of Western irrigation developments water was clearly provided to supply a local need in agricultural products for the margin between the cost of production and returns therefrom, at that time. It was simply a case of values then possible between demand and supply. The resultant of those two general forces constitutes the same basis now and will hold the forces making all future water values. Formerly, this basis of valuation was not affected by the many factors now involved but with the great growth and expansion of the country and its agricultural industry it can be readily seen that water values now are not such a simple matter. They have become quite complex. It is

* Cons. Engr., San Antonio, Tex.

† Received by the Secretary, May 14, 1928.

suspected that every hydraulic engineer has come sternly face to face with naked water values and has been disappointed in his search for information, so the courage of Mr. Field in opening the large subject should not go unmentioned.

Water is one of the natural resources of the country. It is indispensable for plant and animal life. Under the energy from the power station in the sun it forms the transportation system of all plants. It is continually moving, through evaporation and precipitation, and those phenomena constitute the fundamental water cycle in Nature where they are equal and opposite in direction. There is no permanent loss of water anywhere and Nature has provided its purpose and use. Like in all other natural resources, Man is undertaking to obtain from water its maximum use and value. In its extended use additional values are created, just as certain as for the soil or the minerals. As soon as the first additional value is made ownership will attach. Public ownership is natural and essential in the outset for water, just as it was for land, but there is no more reason why water should forever remain the property of the public than it is for the land or the minerals to be common property forever. No section of this country has escaped the inevitable result, yet Colorado is the only State which has shown the courage to clearly recognize it and openly declare water subject to private ownership. It is natural, therefore, that Mr. Field, a former State Engineer for Colorado, should complacently open the unsettled subject without ever mentioning ownership. A peculiar situation thus presents itself and it is predicted the discussion will be watched with interest.

Within the general plan of Nature, it appears that two fundamental natural resources are necessary for the production of crops—climate and soil—and they have been properly related in Nature for the production of each particular crop. The understanding of the proper correlation of all the factors in this combination constitutes the science of agriculture. This is perhaps the oldest science, but it still appears that Man knows very little about it. Present knowledge places temperatures and rainfall as the most important factors in climate and holds that these fix the natural locations for the growth of each crop. Thus it is that wheat, corn, cotton, rice, cane, or apples, peaches, oranges, bananas, or strawberries, blackberries, coffee-berries, or the various nuts, vegetables, oils, drugs, and woods appear with a distinct zone for their natural and economical production.

Under irrigation there is obtained the control of moisture for the growing of plants, and similar soils are found in each temperature zone, and yet crops are limited under irrigation; so it must be concluded that temperatures, which measure the heat energy available, are the controlling factor in Nature's crop zoning. Apparently some crops require a large amount of heat energy for their growth; others are suited for less heat; some want it between certain low limits, others between higher limits, and still others between extremely high limits and can stand no low limit at all. Such limitations appear to fix their production areas, provided the other factors of water and soil are suitable.

The writer believes that a definite and fixed general relation exists between the heat energy required for reproduction of a crop and the water require-

ments for that crop. It is further indicated in Nature that the same amount of heat energy requires the same amount of moisture supply in all plant production. Under that theory it would take the same amount of heat energy and water supply to produce a crop of oats in Texas, Colorado, or Alaska, all other factors except time being equal. The water requirements between crops then depend on their heat requirements. Large heat requirement crops will need large water requirements. High heat requirement crops may not necessarily need large water requirements. The essential factor seems to lie in the initial growing point for the plant as that appears to fix the point in Nature's energy supply which is required to start growth and place it in the zone of temperatures necessary for maturity and reproduction. All this is mentioned here to show that in the last analysis the value of water will lie in each particular kind of crop and the time required for its production to demand value. An acre-foot of water, in the Colorado River Basin, used in Central Park, Colorado, for the growing of native hay, field peas, or perhaps potatoes, all low and short heat requirement crops, will have a certain value, but if that acre-foot were used in the Imperial Valley, for the production of early lettuce, celery, potatoes, or citrus fruits, it would have quite another value, on account of the difference in time of production and kind of production.

The method proposed by Mr. Field for the evaluation of water rights does not go far enough. It is too local, because he bases it entirely on the $6 000 000 estimate given as the value of the Fort Lyon Canal property, but does not show how or why it should be that amount. It is the basic figure which is important and how that was determined, which is essential in the problem; also just how and when that value was created, would be of interest. Furthermore, in the computation used for volume delivered, an average quantity value is obtained, but is not made the final basis of value as the delivery did not cover the full year, and so a correction was made therefor. In that section it is believed water is not used for irrigation during the entire year, so there is no right attached for a full year's diversion and storage rights probably intervene to cover that unused period. It does not appear sound to extend values on that basis and the first value figure obtained by the author stands on a firmer basis than the later one.

It may be of interest to follow the economic steps necessary to create a real water value from its extended use by irrigation. The owners of land, in every semi-arid section of the West, with possibilities of obtaining a supplemental supply of water from a stream, in order to assure full and certain production, have thought, talked, and even taken what action they could, for some step to attach the possibility of a right to the use of some water to their land. A single owner of land soon discovered that he alone could rarely undertake such a task and hence with his neighbors, in mutual organization, the early developments were made. This form of development imposed heroic struggles and has since been supplanted by better means. The history of changes made in the developments by irrigation have been ably recorded and do not need to be repeated here. However, from the very beginning to the present day, the use of the water has been made the basis for a value and its contemplated use made a potential security aspect of considerable importance.

In the outset, for all irrigation properties, the valuation of the water can only be justified on the basis of the cost of the development. Unfortunately, the potential value of water was seen, grasped, and capitalized by speculators, who added it to the land values, and from that practice has grown the troubles from most land settlements under irrigation properties. The real value of water is created by its use and is represented by the actual production of crops the value of which is sufficient to show an attractive margin over the cost of production or over the returns from other competing areas growing similar crops. It does not come into existence until tried and proven use has been made and so should accrue to the party whose work and effort created that value. Any other basis is economically unsound and it appears the other viewpoints have led to much of the present irrigation trouble. Leave the possibility of a reasonable value of the water to accrue to the settler of the land under irrigation, who must make the value, and a changed aspect on most irrigation developments will be noted.

The kind of crops grown and the time of their possible production forecasts the basis for the fundamental difference in the value of an acre-foot of water for irrigation purposes. Any crop grown, on the short-season areas, will come on the market after all other areas of longer season have matured similar crops. For staple crops that handicap may not be so serious. What is perhaps now more depreciating is that such short-season areas are sharply limited in the kind of crop grown so that the areas above them, in the scale of thermal location, contain less hazards and more advantages, which give the latter a better competitive position and thus reduce the value of water in the lower scale area. The oversight of such forces might be pointed to for some of the difficulties inherited by the present Reclamation Bureau. Its recognition then might furnish the basis for a defensible adjustment in their properties, between which relative water values could be determined and these values credited to the user by deduction from the construction cost, when apparently the water value was permitted to be attached. There is little possibility of finding a water value in some properties, with costs of construction and locations known to all irrigation engineers; and this does not mean all Government projects, either.

For some time, the writer has had a growing conviction that the most essential feature in the feasibility of an irrigation development lies in the condition that a clear water value can be reserved for the settler, as compensation and his margin, for the courage, self-denial, effort, and time it will take him to create the value of water for irrigation purposes. If a standard for such a margin could be agreed upon as just in all developments it would soon be quite clear in which zone of thermal location the irrigation developments in the near future would be undertaken.

In the same temperature locations, and under similar crop production, the several areas have a varying demand for water due to the difference of the natural water supply furnished by rainfall. The Imperial Valley of California and South Texas have a very close thermal position and their crops will be much the same; yet with the same consumptive use of water by cropping, which, for illustration, is taken as 36 in. applied to the land, the Imperial Valley obtains little if any direct water supply from rainfall, while South

Texas will average (in the effective support) about 15 in., from its rainfall supply. In one case 3 acre-ft. of water are required to produce the crops, whereas in the other case, 1.75 acre-ft. will produce the same results.

From such practical conditions, which vary for every section, it would seem that a differing value of water exists for each section, and that it does not even remain stationary, but will rise and fall as the margins in the crops produced fluctuate.

FRED C. SCOBEY,* M. AM. SOC. C. E.—In his discussion of the value of reservoir rights the author does not make a distinction that appears essential. A great difference in the right to obtain water from a reservoir lies in the terms of the contract. Does the right entitle the holder to certain space in the reservoir or does it entitle him to definite quantities of water from the reservoir? In California and elsewhere, a number of contracts have been made in recent years whereby one organization agreed to pay to another a certain amount per acre-foot for water delivered—not space in the reservoir but tangible water.

On the other hand a favorite form of agreement entered into several years ago, particularly in Colorado, entitled the purchaser to a certain number of acre-feet of space in the reservoir. If full storage was obtained in any one year the presumption was that water to the extent of the right was started flowing from the reservoir to the lands of the purchaser, who stood the conveyance losses en route. If full storage was not obtained in any year the available water was pro-rated. The space in one well-known reservoir on the South Platte River was sold on the basis of a capacity of about 85 000 acre-ft. As constructed, and as further limited by action of the State Engineer, the reservoir has never had an available capacity of more than about 30 000 acre-ft. Here, the available capacity was only about one-third the assumed paper rights. In "short" years this is, of course, further reduced. Finally, what appeared to be a good and sufficient water right dwindled to merely a supplementary right.

Another case is that of an organization in Utah that sold to another some space at the top of its reservoir. Moreover, it was a space that had seldom or never been filled. In cases like this the numerical volumes of prospective water may be such that a right looks good and sufficient to a land settler, and he does not find that space and water are not synonymous until he fails to receive the latter. When evaluated a distinction must be made.

There are, of course, many reservoirs that receive a full supply practically every year. For these, the consumer's rights need not be discounted. However, there are also reservoirs for which rights have been sold, that are worth only a small part of the same right if it could obtain the water numerically described therein.

LYMAN E. BISHOP,† M. AM. SOC. C. E.—Water rights in the Antero Reservoir near Denver, Colo., entitle the holder to a *pro rata* share of the quantity of water stored each year. The farmers under the Highline Canal own and

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† Cons. Civ.-Hydr. Engr., Denver, Colo.

control 11 254.6 acre-ft. of storage capacity rights in Antero Reservoir. These rights were sold to the farmers by the Antero and Lost Park Reservoir Company and were outstanding when the City of Denver purchased Antero Reservoir and the unsold water rights in it.

The Antero Reservoir water rights of the Highline Canal farmers were in litigation for several years. The Colorado Supreme Court in its decision, interpreting and deciding the value of these oustanding rights to the holders, stated in part:

Farmers have "right to share upon a *pro rata* basis in the capacity of the reservoir in acre-feet as such capacity becomes available * * *.

"* * * a right to use such proportional part of the water as might be stored with an agreement * * * to use reasonable diligence in filling * * *. * * * the contract nowhere provides that they are to receive an acre-foot of water per acre each year out of the reservoir.

"* * * the rights of the farmers under their contracts can be no more than the right to use, on a *pro rata* basis, whatever water may be stored in the reservoir".

It should be noted that the Court states "water as might be stored" and "whatever water may be stored" and does not state or use the terms "is stored" or "is in storage" at any time or year. The City of Denver and the farmers are each entitled to their share and undisputed use, enjoyment, and ownership of their respective capacities. Each may use his stored water; each is entitled to his *pro rata* share of the water that "may be stored" at any time; each can accumulate and hold over his storage; and, unless (or until) all the capacity that belongs to either is filled with water, there has certainly been no damage done to the other. Most assuredly, under Colorado irrigation laws, neither the city nor the farmers will be penalized for conserving water in storage any year and then be required the next year to *pro rata* or divide what has been saved with the other who has used his previous year's storage supply.

The Antero Reservoir as constructed had a capacity of 58 601 acre-ft. At present, on account of the weakened condition of the dam, concrete facing, etc., the State Engineer has limited storage to 33 276 acre-ft. Therefore, the "Highline farmers" who own water rights in this reservoir now control $\frac{11\,254.6}{33\,276.0} = 33.82\%$ of the total water stored in Antero Reservoir each year and the City of Denver owns $\frac{22\,021.4}{33\,276.0} = 66.18$ per cent.

The reservoir was completed in 1909 and water was first stored in May of that year. The City of Denver owns 47 346 acre-ft. and the farmers own 11 255 acre-ft. of Antero Reservoir's original constructed capacity. The reservoir has a decree of date, October 8, 1907, in the amount of 85 564 acre-ft. Of this decreed capacity, the farmers own 11 255 acre-ft. and the City of Denver owns 74 309 acre-ft. Irrespective of its usable capacity, the farmers under the Highline Canal holding Antero water rights own the full amount of 11 254.6 acre-ft. of storage capacity rights. If the reservoir were restored to its original constructed capacity of 58 601 acre-ft., the City of Denver would

then own 47 346 acre-ft. of storage capacity rights instead of the 22 021 acre-ft. now controlled; and if the dam were raised to provide for an additional 6 ft. in depth of storage above the present constructed full supply level, the city would then own and control 74 309 acre-ft. of the then constructed and decreed capacity of 85 564 acre-ft. The date and the amount of the decree that has been entered for Antero Reservoir (October 8, 1907, and 85 564 acre-ft.) are of considerable importance. The storage decrees for Barr Lake enlargement, Prospect Reservoir, Horse Creek Reservoir, and Milton Lake are all of later date than the decree that has been entered for Antero Reservoir. This is quite significant because, since this decree is already entered and is a final one, the raising of Antero Dam to permit the storing of the decreed capacity of 85 564 acre-ft. would result in placing the demands for Antero storage ahead of the demands of Barr, Milton, etc. The South Platte River is the source of water supply for all these reservoirs.

Under the joint ownership and operation by the City of Denver of Antero Reservoir and Lake Cheesman the possible storage available for Antero Reservoir has been estimated in detail by the speaker for each month of the years 1900 to 1925, inclusive. This estimate of possible storage reveals the following as available annually for Antero Reservoir:

Mean (26 years)....................	20 515 acre-ft.
Maximum (1914)..................	58 601 acre-ft.
Minimum (1908)..................	3 845 acre-ft.

The records of Antero actual annual storage since 1909, show the following:

Mean (19 years)....................	13 214 acre-ft.
Maximum (1918)..................	27 801 acre-ft.
Minimum (1911)..................	3 842 acre-ft.

From the foregoing brief statements it will be realized that the value and probable yield of Antero water rights are subject to several criteria. The usable capacity of the reservoir; the variation of the available gross storage; and the requirements and use of the stored water by the water-right owners, are the three principal arguments. All these considerations are subject to change and all changing results in variable yilds for Antero water rights and correspondingly different values.

R. L. Parshall,* Assoc. M. Am. Soc. C. E.—If there is some basis on which it could be said that a second-foot of water has a particular, definite, fixed value and that there is some relation existing between the value of a direct flowage right and storage right, it may be of interest to know that approximately 3 000 acre-ft. of stored water in the Arkansas Valley, in Colorado, was recently purchased at $2 per acre-ft. In the northern part of the State, near Fort Collins, the value of stored water varies according to supply and demand. As a minimum, water in storage sells from $40 to $60 per 1 000 000 cu. ft., while, at other times, the price is as much as $200. In June, 1927, stored water was sold for about $95 per 1 000 000 cu. ft., which is, roughly, $4 per acre-ft.

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PAPERS AND DISCUSSIONS

ADMINISTRATIVE WATER PROBLEMS

A SYMPOSIUM

Discussion*

BY MESSRS. M. C. HINDERLIDER, R. I. MEEKER, LYNN CRANDALL, AND E. B. DEBLER.

M. C. HINDERLIDER,† M. AM. SOC. C. E.—Unquestionably the greatest resource of the arid States of America is their water supplies, and hence the administration problems in this connection are of profound interest to the people of these States. The officials charged with the administration of the Court decrees relating to water supplies, are confronted with many intricate, illusive, and difficult problems.

The laws of most of the arid States relating to the administration of water supplies, lay down broad, comprehensive rules applicable in general to the entire State. Due to the variable character of stream flow, climatic conditions, and the diversity of use of the waters, many conflicts have occurred in the past and others continue to arise to aggravate and confound the officials. Although in general these laws have been interpreted by the Supreme Courts of the various States, there are many shadings in such opinions which of necessity leave to an administrative official many intricate questions on which he must pass in administering the duties of his office.

Colorado, next to California, has the largest area of land under irrigation, and has by far the most extensive system of reservoirs and canals. For the most efficient and economical use of her available water supplies, it has been found needful to provide for a comprehensive system of administration which involves the storage of water in reservoirs located both on the channels of natural streams and at points sometimes far removed from such streams; and for a system of exchange between ditches, reservoirs, and the streams.

* This discussion (of the Symposium on Administrative Water Problems, presented at the meeting of the Irrigation Division, Denver, Colo., July 14, 1927, and published in April, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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The Constitution of the State of Colorado provides that priority as to time of use shall constitute the better right as between those taking water for a like purpose. Furthermore, it is stipulated that the right to divert water from any natural stream for a beneficial use, shall never be denied. No limitation whatsoever is placed on any one who appropriates water on any stream, except that of priority of original use and beneficial application.

Colorado has substantially 1 000 storage reservoirs, in which a large proportion of the available water supplies are stored in times of plenty for use in times of shortage. In order that it may be made available to the various canals and ditches, the law provides that water may be released from reservoirs and transmitted down natural streams and taken out again by the water owners, all under the supervision of the State water officials. This one phase alone involves one of the most difficult and illusive problems with which administrative officials are confronted.

The law provides that the same quantity of water released from a reservoir on a stream above, may be diverted by a canal below, minus a certain quantity for loss in transit to be determined by the State Engineer. This loss varies greatly from time to time, depending on the distance between the reservoir and the canal taking the water; the stage of the river and the quantity of water released by the reservoir; the climatic conditions which may prevail at the time such water is being released from the reservoir; the character of the stream channel, etc. Probably one of the major factors affecting such deliveries is the control of the head-gates of ditches along the stream during such runs of water.

The water officials in Colorado have made many attempts to determine the probable loss of reservoir water in transit under changing conditions, and have found that, due to the variable nature of the many factors involved, it is possible only to ascertain the amount of the losses within certain limits. Such experiments have shown that these losses of water in transit vary from 2 or 3 up to 30% of the water released from a reservoir.

For a proper accounting of all the water supplies in Colorado, the law provides that every reservoir shall be carefully contoured to determine the capacity thereof; and that the reservoirs must be equipped with proper gauge rods, marked in feet and tenths of feet, to indicate the depth of storage therein at any time; that proper measuring flumes or other devices equipped with automatic registers, shall be installed on all streams flowing into the reservoirs and also on all outlets.

The quantity of water in storage at the end of each month in all the major reservoirs is reported to the principal administrative official for that division. While Colorado laws do not divide the calendar year into seasons for direct irrigation and for storage of water, climatic conditions practically effect the same result; that is, when water ceases to be needed for direct application to the land for irrigation, the reservoirs are permitted to store water in order of priority.

When stored water is required by owners along the various streams, it is released from the reservoirs under supervision of the local water officials, measured over automatic registers, and a similar quantity of water deliv-

ered to the head of the canal below, minus a certain quantity which is deducted for loss in transit.

Assuming that all head-gates located on the stream between the reservoir and the point of delivery are properly regulated and controlled to insure that other owners do not divert water to which they are not entitled, the only material loss in transit is occasioned by evaporation.

It is true that under certain peculiar conditions loss of water from a stream occurs through subterranean channels which may lead to lower areas adjacent to a stream, but such losses are rather infrequent except for short stretches along a stream.

Some time ago the speaker conceived the idea of attempting to compute evaporative losses from stream channels by first ascertaining the exposed surface area of the channel at different stages of flow, and then applying the proper evaporation coefficient applicable for any particular day. It was soon discovered, however, that the stage of the stream varied radically both day and night, especially in shallow streams with sandy beds and numerous sand-bars. This made it impossible to determine with any reasonable degree of accuracy the probable surface area exposed to the air and sun at any time.

As a means of securing more efficient administration of reservoir runs, it was found needful to exercise the most painstaking supervision over the head-gates along the river between the points in transit. To effect such administration it was necessary to require that automatic registers and accurate measuring devices be installed in the ditches. With such equipment supervised by the local water commissioners and their deputies, it has been found that many of the old problems affecting the transmission of reservoir water have been successfully met.

There are eight requisites for a proper and efficient administration of both canal and reservoir decrees and for the transit of reservoir water along a natural stream. These may be listed as follows:

First.—Proper head-gates in all ditches heading on the stream, with facilities for locking them down if necessary.

Second.—Proper measuring devices equipped with automatic registers for determining the quantity of water diverted hourly by the various ditches.

Third.—Numerous gauging stations equipped with automatic registers placed along the stream at strategic locations.

Fourth.—An efficient corps of experienced hydrographers to rate the gauging stations and canals and patrol the river during reservoir runs.

Fifth.—Conscientious water commissioners and deputies to patrol the stream and the head-gates of all canals to see that the water is distributed strictly in accordance with order of priority.

Sixth.—A proper system of telephone communication between the reservoirs, the river stations, all canal head-gates, and the chief administrative official.

Seventh.—A proper system of daily reports between the local water officials and the chief administrative official, showing the quantity of water (both morning and evening) available for distribution; the quantity of water

diverted by each head-gate; the quantity of water stored in each reservoir, and in transit down the stream for any ditch.

Eighth.—A proper register of the Court decrees showing the order of priority in which waters may be diverted or stored, and a thorough understanding of these decrees by the administrative officials.

As a further material aid in the administration of reservoir runs and other water decrees, State officials have found that a diffusion of the knowledge of what occurs on the stream each day, or possibly more frequently, by means of a bulletin issued by the chief administrative official or secretary of the ditch organizations along the stream, is most beneficial.

They have found that a complete knowledge of water supply demands and distribution, if made available to the water user, dissipates suspicion, removes grounds for complaint against the administrative official, and provides a permanent record for future reference, which is invaluable.

As time passes and water supplies become more valuable and require a more careful supervision and greater efforts toward conservation, greater efficiency will be demanded, which will inevitably result in improved methods of measurement, control, and accounting.

Mention has been made of gains to the river resulting from the transit of reservoir water along a stream. There can be no actual gain or accretion in the total available water supply as a result of such reservoir runs, but, on the contrary, there must always result some loss of the water in question, due to evaporation and, possibly, to deep percolation away from the stream channel.

It is true, however, that following these reservoir runs, the stream frequently benefits because of the return flow of percolating waters draining out of the sand-bars and land adjacent to the stream channel.

If the percentage of assumed losses chargeable against the reservoir is excessive, the reservoir run suffers. If on the other hand the estimated percentage for loss of water in transit is not sufficiently high, the other rights along the stream suffer.

After releasing water from a reservoir farther up on a stream, and turning an equivalent quantity of water out of the river into the head-gate of the receiving ditch, it frequently happens that some other canal immediately below, which up to that time had been securing its full quota of water, is deprived of all, or some portion, of the water theretofore drawn.

Unless there has been a material change in climatic conditions affecting the natural stream flow, it is obvious that the reservoir run has not been sufficiently penalized, with the result that great injury is done some water user along the river. Since the values of these water supplies are measured daily in dollars and cents, the effect of poor administration has all the results of depriving an owner of actual property with all the resultant dangers of ill-feeling, litigation, property losses, etc.

From the foregoing it may be observed that the intricacies involved in the administration of the water supplies in the Western States and a proper solution of the problems, challenges the earnest and intelligent effort of the administrative official, the best statesmanship of legislative bodies, and the most painstaking thought of the Courts.

R. I. MEEKER,* M. AM. SOC. C. E. (by letter).†—The author has tersely presented his administrative experience dealing with transmission losses incident to runs of several thousand second-feet of reservoir water for stretches of several hundred miles in a large river, in which channel conditions as to gains and losses are variable and administrative problems are complex.

The history of river administration should record an evolution in charges imposed on reservoir water conveyed in river channels. Realization on the part of water users and water officials that there are wide fluctuations in river gains and losses due to stream-bed formations, return flow from irrigation, and other causes, will go a long way toward securing the necessary funds to make engineering studies with which to guide administrative practice.

Continued engineering measurements of river channel flows for gains and losses are necessary in order to fit administration practice to conditions of natural fluctuating river stages, to conform to changing river conditions in irrigated valleys, and to eliminate conjectures and prejudicial assertions frequently made by opposing parties in such matters. Channel losses or gains in a river system may be far different to-day than those of the past or the future. They also vary with the size and compactness of adjacent irrigated areas.

By statutory enactment in 1879, conveyance of reservoir water in river channels in Colorado was made subject to deduction for evaporation and seepage losses. Subsequent legislative acts in 1897 imposed a "reasonable deduction for seepage and evaporation" on waters transferred from one stream to another, or on exchanges of reservoir and ditch water. The State Engineer was designated as the agency to make determinations of losses applicable to such waters.

In framing legislation concerning conveyance losses of reservoir water, inter-water-shed diversions, and exchanges of water, the underlying principle has been "the non-injury to other water rights upon a stream system." In administrative practice, reasonable doubt, therefore, has usually been resolved against the reservoir water.

Notwithstanding engineering information on river channel losses, the question of penalty losses applicable to reservoir waters transferred in river channels is fertile ground for water disputes, especially in dry years when the effect of water shortage is felt. The writer's experience on losses applicable to reservoir water in transit in river channels convinces him that frequently grounds for complaint by direct-flow users are justified. Actual losses sustained by prior users sometimes occur from weak administration of intervening diversions, in river sections between the reservoir and the diversion works, rather than from actual channel losses by seepage. Evaporation losses of water in transit usually are negligible when compared to percolation or administrative losses. Seepage and administrative losses are frequently confused.

In mountainous country, channel percolation losses are usually quite small. In plains and valley sections of rivers, both seepage and administrative

* Cons. Engr., Denver, Colo.

† Received by the Secretary, May 10, 1928.

losses may be heavy. River channels through lava (basalt formations) are likely to be erratic as to losses or gains, and should be evaluated by actual measurements. As river channel losses are seldom directly comparable, the application of percentage deductions for one river section to other river sections, or from one river channel to another, is dangerous.

On account of lack of funds for engineering studies in advance of reservoir runs, transmission losses applicable to "reservoir runs" generally have been set by stipulation of the interested parties and water officials, and, later, have been checked by engineering measurements and modified in accordance therewith. The time has come for engineers, water users, and administrative heads to insist on funds for investigating these matters. The loose haphazard methods of the past concerning penalty losses applicable to reservoir and interstream waters should be relegated to oblivion. Problems of dwindling water supplies, increasing use of stored waters, and changing river conditions due to irrigation, power, and industrial uses, demand engineering treatment.

TABLE 2.—Transmission Losses Imposed on Reservoir Water.

Reservoir.	Stream.	River channel distance, in miles.	Character of river bed.	Deduction, percentage.	Basis of determination.
Twin Lakes....	Arkansas......	160	Chiefly through mountains.	10	By stipulation; engineering studies under way.
Rio Grande....	Rio Grande....	80	Through mountains.	8	By stipulation.
Antero.........	South Platte ..	100	Through mountains.	12.5	Stipulation modified by engineering studies.
Cheesman	South Platte ..	25	Through mountains.	2.5	By stipulation and stream-flow records.
Cheesman	South Platte ..	55	Through mountains and plains.	5	By stipulation and stream-flow records.

Table 2 outlines roughly the transmission losses charged against reservoir water in Colorado, and data therein contained are taken from records of the State Engineer's Office at Denver.

Lynn Crandall,* M. Am. Soc. C. E. (by letter).†—The construction and operation of storage reservoirs on natural stream channels in the irrigated areas of the Western United States have resulted in more or less contention between the owners of the stored water and the owners of earlier natural flow rights. The disputes thus occasioned generally pertain: First, to the segregation of stored water and the natural flow as discharged from the reservoir; and, second, to the proper transmission loss to be charged to stored water while in transit in the stream channel between the reservoir and the place of diversion from the stream.

The practice of determining the daily amount of storage released from reservoir drop and capacity tables, as used at Jackson Lake and Henry Lake

* Water Commr., Big Lost River, Mackay, Idaho.

† Received by the Secretary, June 13, 1928.

Reservoirs and elsewhere, is probably the simplest method to apply. In all cases it is useful as a check on other methods, but its application causes all errors and fluctuations from erroneous capacity tables, evaporation losses, return ground storage, etc., to be thrown into the determined daily natural flow. This, in turn, causes fluctuations in the determined natural flow from day to day that would not exist except for the construction and operation of the reservoir and the use of this method of segregating the stored water from the natural flow. While such fluctuations may balance throughout the season, the natural flow rights are often of varying dates of priority, and during periods when the stream flow is not sufficient to fill all the natural flow rights, this method may deprive certain of such rights of water, to the advantage (at some other time during the season) of other natural flow rights of different dates of priority. For example, in various reservoirs with which the writer is familiar the water held as ground storage amounts to from 3 to 10% of the open reservoir storage as determined from the contour surveys, and during the course of the season this may compensate fully for evaporation losses. However, such losses are generally greatest when the reservoir is full, while the return from reservoir ground storage usually reaches a maximum later in the season when the reservoir is being lowered rapidly. Under this method of operation, then, the evaporation loss may cause a certain natural flow right of late priority to be cut off earlier than it should be, while the ground storage gain accrues to an earlier right still being filled later in the season. On reservoirs with numerous fluctuating sources of supply, however, this method is about the only practical one, but in such cases its use should be supplemented by special investigations as made at Jackson Lake. Records of all inflow, outflow, evaporation, ground-water fluctuations, etc., should be secured so as to afford a basis of evaluating the effect of operation on any particular rights involved, as well as a basis for calculating a reservoir capacity table, based on inflow and outflow records, independent of the capacity table determined from contour surveys.

Where there is no natural gain to the stream in the reservoir site, or where such gain, if any exists, comes from springs that are fairly uniform in flow, it will usually be found more satisfactory, fairer to every one concerned, and less productive of dispute, to determine the reservoir inflow each day from a gauging station, or from stations situated above the back-water of the reservoir. Then add to or subtract from such inflow an agreed or determined flow for natural gain or loss in the original stream channel through the reservoir, thus securing the natural flow passing the gauging station below the reservoir, any surplus at that point, over such determined natural flow, being deemed released stored water. This method is prescribed in the Court decree governing the distribution of the waters from Big Lost River, Idaho. An allowance of 34 sec-ft. is made for natural stream gain in the Mackay Reservoir, which is based on records compiled during periods when the reservoir was empty. A similar method is followed in the operation of the Magic Reservoir on Big Wood River, Idaho. On Big Lost River, it has been found necessary to locate the measuring station at least 2 miles below the dam, in order to get far enough down stream to intercept all the ground-water outflow from the reservoir,

more than 100 sec-ft. of which flows around or under the dam itself when the reservoir is full. Even in the case of reservoirs with a large inflow from springs (provided records are available prior to the reservoir construction or during periods when it was empty), it may be more satisfactory and equitable to determine or agree on a figure, to represent such natural gain through the reservoir site. If the records justify it, a slightly different figure should be used for different times during the season, rather than rely upon some method which might result in unwarranted fluctuations being caused in the calculated natural flow.

Having segregated, by some means or other, stored water from natural flow as it leaves the reservoir, the question of proper transmission loss to charge stored water as it flows in the natural channel, mingled with natural flow, to the point of diversion from the stream, still remains as a source of contention. In this case, as in the case of segregation of water at the reservoir outlet, it would seem fair to start with the basic assumption that the natural flow rights are entitled to the quantity of water that would be available at their points of diversion—if the reservoir had never been built or if no stored water had ever been carried in the stream, and that any additional amount available over and above such quantity belongs to the owners of the stored water.

The loss occasioned by running stored water in natural stream channels will vary widely, depending on whether the stream naturally loses or gains water in the section considered. The head-water areas of most Western streams are regions of fairly heavy precipitation, where the stream gains from tributary ground-water inflow. As the stream emerges from the mountain canyons into the lower valley, it often flows for some distance over a delta or cone where the adjacent ground-water level is lower than the water level in the stream, and where the natural channel loses water at all stages. Farther down stream the ground-water may again rise higher than the water level in the stream, resulting in a ground-water inflow, and it may then gain to the stream in such sections. On numerous Western streams such alternate sections of gain and loss occur for many miles.

In sections where the stream loses water due to the adjacent water-table being lower than the level in the stream, the stored water should manifestly be charged with the additional loss created by carrying it, as compared with what the natural loss would be without including the stored water. The practice of determining percentage losses in the channel with both stored water and natural flow in the stream, while supplying information of interest, should not be relied on entirely to determine proper losses chargeable to stored water. A comparison of losses with natural flow alone, and when carrying the additional stored water should also be made. Studies of the ground-water flow through the valley must necessarily be made to determine whether the water lost in the "loss" sections of the stream returns farther down stream as gain and, if so, when and where such return takes place. Stored water is properly entitled to credit for whatever portion of the "lost" stored water that returns to the stream above diversion points during the period of full use of the stream flow each year. In some of the larger river valleys of the West, this "lost" water requires several years to complete its underground travels and finally

emerge as ground-water or spring inflow into the stream. Thus, the continual carrying of stored water for year after year may result in that portion of such stored water that is lost in the "loss" sections, finally re-appearing as a permanent increase in the gain to the stream in the "gain" sections.

In sections of the stream where the water-table is higher than the water level in the stream channel, the stored water sustains little actual loss, but it has the effect, when first turned in, of damming back part of the natural ground-water inflow until the ground-water slopes adjacent to the stream re-adjust themselves to the higher water level in the stream occasioned by the stored water. The effect of running stored water in such sections of natural gain from ground-water inflow to the stream can best be studied by noting the reduced gain for a short period, varying under different conditions from a few days to several weeks, when stored water is first turned into the channel. This temporary bank storage returns rapidly to the stream when the stored-water run ceases and the stream drops to its natural level. In the delivery of stored water on Big Lost River, the matter is adjusted by allowing the natural flow rights to use some of the stored water during the first week that it is turned into the stream, the same quantity being repaid to the stored-water owners later in the season from the increased gain that occurs for a brief period when the stored water run ceases.

As pointed out by Mr. Baldwin,* the matter of the equitable segregation of stored water and natural flow is a complicated one in many cases. It is not subject to exact determination on account of differences in climatological factors, ground-water levels, stream channel conditions, etc., from year to year and at different times during the same year. By securing records covering a number of years on inflow, diversions, and losses along the stream, together with studies of the ground-water movements, however, a basis may be developed for an intelligent study of the question, and a fair approximation may be thus reached for the determination of the natural flow and stored water, as each is delivered to its respective owners.

E. B. Debler,† M. Am. Soc. C. E.—The State of California adopted riparian rights as the basis of its water rights many years ago, and as no affirmative action on the part of the grantee was required, such rights became vested as to all riparian lands, with the action of the State Legislature.

Fundamentally, riparian rights imply non-consumptive uses only, but in the main the Courts have upheld any uses which did not deprive other riparian owners of their proportionate share of such use.

Recent attempts to impose limitations on the use of these riparian rights, even if only in support of a more efficient use of the State's water resources, either by legislative enactment or by regulations of an administrative arm of the State Government, are likely to prove fruitless as they would appear to be in the nature of confiscation of a vested property right which the Courts will not sanction. In view of this situation, plans for the conservation of the water resources of the State must be placed on a sounder legal basis.

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* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1080.

† Engr., U. S. Bureau of Reclamation, Denver, Colo.

The situation with regard to conflict of riparian and appropriation rights is not limited to California. Both types of rights exist in a number of Western States. Nebraska has adopted riparian rights for its humid section and appropriation rights for its arid section. Land grants by the Spanish Crown in the Southern Border States often carried water rights, sometimes distinctly "appropriation" in character, at other times, "riparian." The outcome of California's present attempts to curb riparian rights will be watched with interest, as it is likely to be followed in after years by similar attempts in other States when water becomes of greater value.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

RETURN WATER AND DRAINAGE RECOVERY FROM IRRIGATION

A SYMPOSIUM

Discussion*

By Messrs. R. I. Meeker and Fred H. Tibbetts.

R. I. Meeker,† M. Am. Soc. C. E. (by letter).‡—The paper by Mr. Willis on return-flow waters is a welcome addition to literature on irrigation. It is the first paper to be submitted to the Society on this important phase of water supply. The question of return-flow water from irrigation is recognized by engineers as a large factor in river supplies and it is beginning to be appreciated generally by water users on river systems in all the Western States.

Uncertainties concerning dependence on seepage water from irrigation as a large factor in water supplies, long ago passed from the field of speculation in Colorado, where pronounced return-flow waters from irrigation have developed; where seepage or return-flow measurements were initiated in 1885;§ and where such measurements have been systematically made to date.‖ The increasing volume and magnitude of such waters are a revelation to the water users in the more recently irrigated areas of the North Platte Valley. If head-gate diversions were a total loss to the North Platte River—as used in water supply studies in 1914—the then threatened interstate water shortage of three States would have become a certainty by this time. Present measurements of return-flow or seepage waters in the North Platte Valley, together with recent engineering data on net water requirements of irrigated lands in older irrigated

* This discussion (on the Symposium on Return Water and Drainage Recovery from Irrigation, presented at the meeting of the Irrigation Division, Denver, Colo., July 14, 1927, and published in April, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Denver, Colo.

‡ Received by the Secretary, April 16, 1928.

§ "Seepage and Return-Flow Waters, Pt. 1; General Discussion and Principles," by L. G. Carpenter, Agricultural Experiment Station, Colorado Agricultural Coll., Fort Collins, Colo.

‖ Biennial Reports, State Engineer of Colorado.

valleys, are sufficient bases for the assertion that the water supply of the North Platte Basin is sufficient, when controlled by reservoirs, for the needs of three States, with water to spare.

Return-flow measurements and consumptive-use determinations of irrigated lands supplement each other, go hand in hand, and are destined to play a large part in the future irrigation history of rivers. A non-allowance for return-flow water in engineering studies of river supplies will result in a restricted forecast of acreage of lands which may ultimately be served. The beneficial and healing effects of return flow have turned many an irrigation ditch, initiated prior to the set-up of return flow, from failure to success. Return flows from irrigation usually set up slowly and progressively, and may require from ten to twenty years to reach a stabilized condition, dependent on soil conditions, topography, and water usage.

No better illustration of the benefits to be secured from soil storage of water is to be had than that in Fig. 1* on which the natural run-off of the North Platte River is shown, with September and October low flows increased 200 per cent. In recent years the winter flow of this river at Bridgeport, Nebr., has been chiefly return flow.

In the river section between Whalen, Wyo., and Bridgeport, drainage recovery on the North Platte Project has been a large factor in the heavy volume of return flow shown in Fig. 2.† At the close of 1927 the drainage system of this project consisted of 350 miles of open-drain ditches costing $1 400 000. This project is a large factor in the heavy seepage and return-flow waters of the North Platte River section from Whalen to Bridgeport. Of the 350 000 acres now irrigated in this section, 170 000 acres are lands of the Federal Project. According to records of the U. S. Bureau of Reclamation, head-gate diversions of the project now average almost 650 000 acre-ft. per year.

With present information a correlation of head-gate diversions, return-flow data, and consumptive-use determinations of the North Platte Valley between Whalen Dam and Bridgeport, where 350 000 acres are irrigated, shows the following relations concerning water used for irrigation:

Head-gate diversion, 3.50 acre-ft. per acre per year	100%
Return flow, 2.30 acre-ft. per acre per year	67%
Consumptive use of river water, 1.20 acre-ft. per acre per year	33%

With respect to return flow, the irrigated lands of the North Platte Valley in Wyoming and Nebraska are commencing to function in a manner similar to the irrigated lands of the South Platte Valley in Colorado. On account of the more abundant water supply, with consequent lavish use on the irrigated lands, larger return flows per unit of area irrigated are to be expected in the North Platte Valley.

South Platte Valley Return Flow.—The irrigation history of the South Platte Valley in Colorado, furnishes the confirmatory background of what may be expected as to return-flow waters in the North Platte Valley when

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1099.

† *Loc. cit.*, p. 1100.

irrigation conditions become stabilized. Return-flow measurements have been made systematically in the Colorado irrigated areas for forty years. A recent determination of return-flow waters to the channel of the South Platte River (in a distance of 236 miles, extending from a point at the foothills 20 miles above Denver to Julesburg, Colo.) shows a gain of 6 sec-ft. per mile of channel, a total gain of 1 475 sec-ft., which is equivalent to a yearly return of 960 000 acre-ft. These figures apply only to the channel of the river and do not include the return-flow waters of tributary streams, such as Bear and Clear Creeks, St. Vrain, Big Thompson, and Cache la Poudre Rivers, on which large acreages are irrigated. All these streams are recipients of return flow in considerable quantities, and, although such waters are re-diverted and re-used within their own basins, considerable volumes of return flow annually reach the channel of the South Platte River. Such tributary inflow is excluded from river gains.

As an illustrative example of the beneficial and healing effects of return-flow waters, the changed status of the Lower South Platte Valley, from Kersey to Julesburg, is cited: 240 000 acres are now irrigated in this section, where, in 1902, only 100 000 acres were irrigated. This region depends almost exclusively on return flow for its water supply. Five reservoirs with a total capacity of 275 000 acre-ft. store and control the winter return flow. In 1902, the Lower South Platte Valley was considered the poorest section of the South Platte River for water supply. To-day, water conditions are just the reverse. In 1925, a low-water year (60% on mountain run-off), the junior ditches and reservoirs of the Lower South Platte Valley received an adequate water supply and the best supply of any section of the South Platte Basin, whereas upper areas, under senior ditches dependent on direct-flow water from the mountains, suffered water shortages.

Table 1 shows in a comparative way the progressive increase of return flow to the main channel of the South Platte River from the head of the irrigated area to the Colorado-Nebraska line, as determined by return-flow measurements covering a period of 35 years. The table was prepared from records of the State Engineer's Office of Colorado.

TABLE 1.—RETURN-FLOW WATER TO MAIN CHANNEL, SOUTH PLATTE RIVER, PLAINS AREA, EASTERN COLORADO, FROM PLATTE CANYON TO JULESBURG, COLORADO, 236 MILES OF RIVER CHANNEL.

Year.	Total second-feet.	Acre-feet per year.
1891	580	375 000
1900	880	570 000
1908	1 200	775 000
1916	1 410	915 000
1926	1 480	1 000 000

Evidence of the large return flows of the South Platte Basin and consequent re-use of water may also be best indicated by the statement that with an average inflow of mountain water of 1 500 000 acre-ft. per year, 1 100 000 acres

are irrigated, and still the unused residue passing to Nebraska is 400 000 acre-ft. per year. A large return flow from soil storage with re-diversion and re-use several times is the answer.

Literature on Return Flow.—The following literature on return flow is cited for convenience of engineers who have need of information on this important subject:

"Seepage or Return-Flow Waters from Irrigation", by L. G. Carpenter. *Bulletin 33,* State Agricultural Coll., Fort Collins, Colo., 1896.

"Seepage Water of Northern Utah", by Samuel Fortier. *Water Supply Paper No. 7,* U. S. Geological Survey, 1897.

"Irrigation in Arizona", by R. H. Forbes. *Bulletin 235,* p. 51, U. S. Dept. of Agriculture, Office of Experiment Stations, Washington, D. C., 1911.

"Irrigation in Colorado", by C. W. Beach and Porter J. Preston. *Bulletin 218,* U. S. Dept. of Agriculture, Office of Experiment Stations, Washington, D. C., 1910.

"Seepage and Return Waters", by L. G. Carpenter. *Bulletin 180,* Pts. 1, 2, and 3, Agricultural Experiment Station, Colorado Agricultural Coll., Fort Collins, Colo., 1916.

Biennial Reports, 10th, 11th, 13th, 14th, and 17th, State Engineer of Colorado; also, unpublished data.

"Irrigation in Northern Colorado", by R. G. Hemphill. *Bulletin 1026,* p. 10, U. S. Department of Agriculture, 1922.

"Return Flow Water from Irrigation Developments", by R. I. Meeker. *Engineering News-Record,* July 20, 1922, pp. 105-108.

"Return of Seepage Water to the Lower South Platte River in Colorado", by R. L. Parshall. *Bulletin 279,* Agricultural Experiment Station, Colorado Agricultural Coll., Fort Collins, Colo., 1922.

Division of Water Rights, Pt. IV, 1922, Department of Public Works, State of California, pp. 49, 98, 99, 104-110; Pt. III, 1924, Return Waters—Sacramento and San Joaquin Rivers, pp. 131-139.

Biennial Report, 15th, State Engineer of Wyoming, Supplement B, "Seepage and Return Water in Wyoming Streams During 1920", by Robert Follansbee, pp. 82-96.

FRED H. TIBBETTS,* M. AM. SOC. C. E. (by letter).†—When an irrigated valley is generally underlaid by a water-bearing formation of substantial vertical dimensions and readily available porosity ("specific yield"), and particularly when such a valley is irrigated in large part from reservoir releases where the accumulation of impounded water is subject to the great annual variation of arid sections, economic studies will probably show that full development under such conditions will indicate extensive occasional use of the underground formation for cyclic storage. This general hypothesis is steadily becoming evident in regions like the Salt River Valley of Arizona and the San Joaquin Valley of California. Deep percolation will gradually fill the water-bearing formation with water which, especially if the outflow at the

* Civ. Engr., San Francisco, Calif.

† Received by the Secretary, September 2, 1927.

lower end is blocked or retarded, will be protected over a period of years from the large evaporation losses of surface reservoirs, and will then be available during seasons of great water shortage when, by pumping from wells, it can be recovered and used. In such cases it will quite probably be found to be more expensive to distribute the water (including the pumping lift) from the underground reservoir than from surface reservoirs. However, inasmuch as the increase in cost is for power for pumping, this item, felt only during the infrequent cycles of particularly dry years, will not affect greatly the average cost of water.

It is likely that an ultimate balance on large sized projects of from 400 000 to 600 000 acres, will be found when from 10 to 30% of the total water supply is obtained by pumping. The Salt River Valley, the San Joaquin River Water Storage District, and the Pine Flat Storage District (Kings River, California) would be included in this class. In cases of emergency when surface reservoir supplies fail there will be available the possibility (by pumping down the water plane) of furnishing a large part of the required water in this way. Pumping operations of this kind may be perfectly feasible, especially in occasional emergencies, if the aggregate depth of available water-bearing formation within the limits of about 300-ft. wells, is as much as 40 ft. of gravel or sand in which the specific yield under field conditions is about 15 per cent. In such a case a total of 3 ft. of water per year is theoretically procurable, for two successive years from this source alone.

As a further contribution to the subject of return water the writer wishes to present briefly a highly specialized case which apparently violates all the rules of orthodox irrigation engineering in that it seems to demonstrate that the more water used the better for everybody.

The rice industry in California, starting in 1912 with 1 400 acres, increased in 8 years to 162 000 acres. In 1919, the rice crop was valued at more than $21 000 000. Although rice growing is the greatest of cereal industries, it has nowhere else been introduced quickly on arid lands requiring large quantities of irrigation water with a relative shortage of cheap labor. In California, where labor is scarce and costly, Oriental methods must be modified so as to require a minimum labor cost in preparing and tilling the land, and particularly in irrigating and harvesting the crops. One method of accomplishing this result is to allow the diversion of excessive quantities of irrigation water. Experience has shown that on the areas best adapted to rice culture, the encouragement of such excessive diversions does not lead to the wastage of water and hence the impairment of irrigation water resources.

Analysis of return water from rice irrigation has all the simplicity of a textbook problem, as all the water diverted can be completely and accurately accounted for. On three typical projects in the Sacramento Valley—one irrigating about 40 000 acres of rice, another about 10 000, and another about 900—the irrigation diversions have been measured since 1919. The waste or excess water from these projects is almost immediately collected in drainage canals and returned to the Sacramento River, and again is accurately measured within a few miles of its diversion point. In the typical rice operations of

Reclamation District 108, in Colusa Basin, where about 10 000 acres are under irrigation, water is diverted from the river by pumping or by gravity, depending on the river stage. It is then returned to the river within 18 miles of the diversion point in the same manner. Fig. 4 shows for each year since 1919, the average diversion per acre irrigated, segregated into the "consumptively used" water and the water immediately returned to the river. The point that the writer wishes to emphasize is that the consumptive use of the water, and hence the quantity extracted from the river, is practically independent of the quantity diverted. The net extraction is equivalent to a depth of about 5⅜ ft., and whatever the diversion from the river the remainder, in excess of 5⅜ ft., is immediately returned.

The drainage water is concentrated in deep drainage canals so that the surface of the water in the canals is generally below the ground-water level during the irrigation season, and always so when adjacent to the rice fields where the land is kept continually submerged. Rice irrigation should be, and in the main is, restricted to heavy soils such as clays and clay-adobes, in which loss of water by deep seepage and percolation is negligible. In soils of these types the seepage is excessively slow. Furthermore, the distance through which the drainage water is transported is so small that evaporation losses are negligible. The result is a clear-cut condition in which the drainage water, or return seepage water, is essentially the diverted water, less the water used in irrigation. The latter represents evaporation losses in the beginning of the season, and when the rice plants are high enough to shade the soil, it represents principally transpiration losses through the growing plants. The average consumption of water from these causes is shown in Fig. 5 for the irrigation season of April, May, June, July, August, and September, in inches per month. This is shown to be much in excess of the normal evaporation losses from a free water surface.

One of the advantages of rice irrigation in California is that it makes possible the utilization of lands formerly considered of little value because of alkali. Even if in the first place the lands were not alkaline, it is clear that keeping them submerged to a shallow depth during the hot summer months would tend to draw up alkali from the subsoil. Careful chemical determinations over a period of years in District 108 indicate that in the quite soft Sacramento River water, the 5 acre-ft. used, contains about 2 000 lb. of alkali ("alkali" being popularly taken as the amount of material in solution). If river diversion is restricted so that waste water is less than 15%, alkali accumulates on the land, but if the waste is equal to about 30% of the diversion, that is, if the waste is equal to one-half the water consumed, then from two to four times as much alkali is removed in the waste water, as is applied in the irrigation water. The actual removal of alkali in one instance was at the rate of 9 750 lb. per acre per year.

An ample water supply permits the use of large irrigating heads and large rice checks, with a substantial reduction in labor cost for preparing the soil and distributing the water. Free flow through large checks keeps the water fresh, and tends to maximum yields and early maturities.

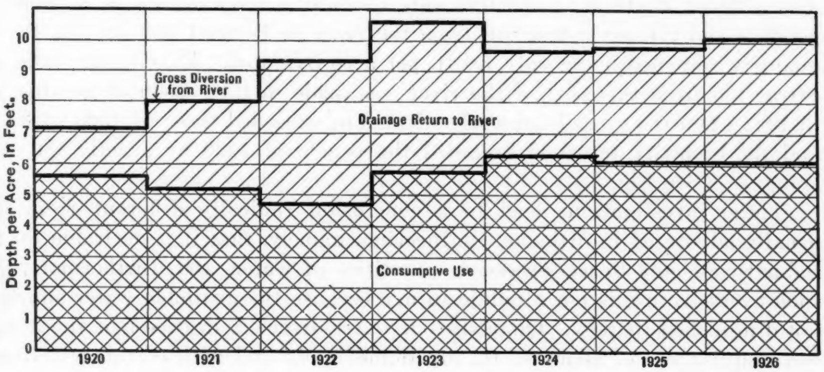

FIG. 4.—ANNUAL DIVERSION, USE, AND RETURN, RECLAMATION DISTRICT NO. 108.

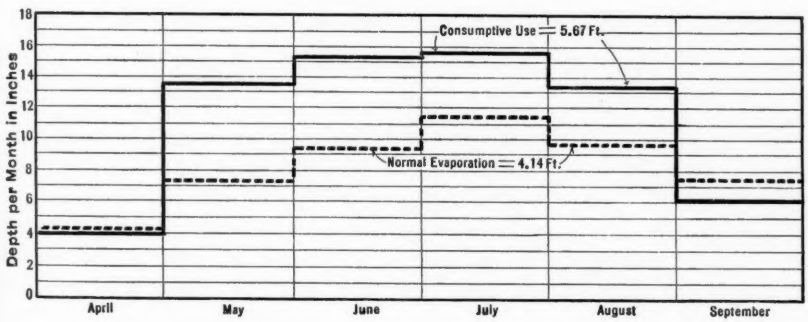

FIG. 5.—AVERAGE CONSUMPTIVE USE AND NORMAL EVAPORATION.

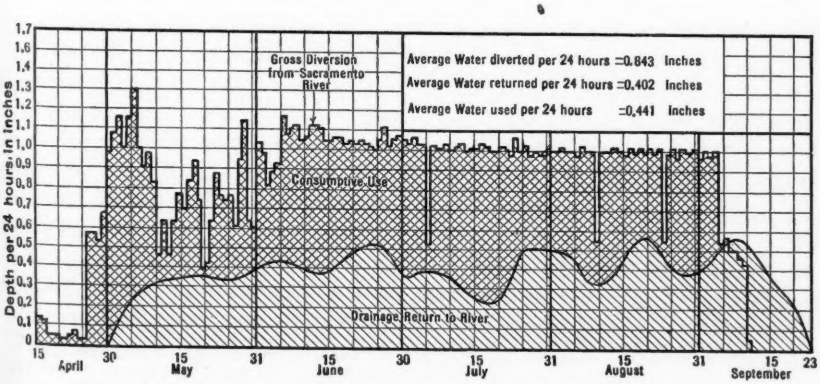

FIG. 6.—TYPICAL SEASONAL DIVERSION, USE, AND RETURN.

In 1920 when an alarming shortage of water was impending in the Sacramento River, State water-masters were restricting river diversions as much as possible and the net extraction from the river in District 108 was 5.7 ft., the quantity returned being only 1.6 ft. per acre irrigated. Exactly the same net extraction from this part of the river was made in the season of greatest use in 1923 when the gross head-works diversions exceeded those of 1920 by 3.3 ft. and the quantity returned exceeded the return of 1920 by exactly the same amount. (See Fig. 6.)

These investigations lead to a conclusion radically different from that usually advocated in orthodox engineering, namely, that it is desirable for the industry affected to use excessive quantities of water, and, furthermore, that such excessive use is not a public injury because the available water supply is not diminished thereby. Quantities of water diverted, in excess of the normal consumptive use of about $5\frac{3}{8}$ ft., are immediately and with certainty returned to the stream for re-use.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

TRANS-MOUNTAIN WATER DIVERSIONS

A SYMPOSIUM

Discussion*

By Messrs. A. Lincoln Fellows and Ralph Bennett.

A. Lincoln Fellows,† Esq. (by letter).‡—Mr. Debler has furnished a valuable contribution to the literature on trans-mountain diversions which the writer takes pleasure in endorsing as generally correct. However, in the interest of historical accuracy he desires to supplement the statement made with reference to the commencement of work on the Gunnison Tunnel. The author correctly states that the plan was conceived in 1890 by Mr. L. C. Lauzon, but the impression is given to the casual reader that the State of Colorado led the way in commencing work on the construction of the tunnel. This is true in so far as actual construction on a State tunnel was concerned; but, in reality, the U. S. Geological Survey was first, by some months. That organization set apart funds for the commencement of surveys in the latter part of 1900, on the recommendation of F. H. Newell, M. Am. Soc. C. E., and the writer. The appropriation by the State Legislature followed in 1901, five or six months later. Construction of a tunnel by the State was begun after the first preliminary survey was completed, before the final surveys and reports were made, and contrary to the advice of the writer. This State tunnel was entirely distinct and located at a considerable distance from the Government location. It was eventually abandoned completely.

Some very interesting trans-mountain diversions have been made in Wyoming, several of them dating back many years. Indeed, Wyoming has been a pioneer in making such diversions. The writer also calls attention to

* This discussion (of the Symposium on Trans-Mountain Water Diversions, presented at the meeting of the Irrigation Division, Denver, Colo., July 14, 1927, and published in April, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Irrig. Engr., Bureau of Public Roads, U. S. Dept. of Agriculture, Denver, Colo.

‡ Received by the Secretary, June 11, 1928.

a notable project in North Dakota—the suggested diversion of the Missouri River into the Red River drainage basin, or, in other words, the proposed diversion from a stream discharging into the Gulf of Mexico to the Hudson Bay drainage basin. The State Irrigation Engineer of North Dakota, Robert E. Kennedy, M. Am. Soc. C. E., has recently prepared preliminary reports regarding this project.

Ralph Bennett,* M. Am. Soc. C. E. (by letter).†—Only those routes for the proposed Los Angeles-Colorado River Canal that lie north of the Imperial Valley, are mentioned specifically by Mr. Van Norman.‡

There are also a number of possible routes in which the water would be diverted through the Imperial Canals and carried in them for varying distances before entering the plants that would lift the water over the range (see Fig. 3).

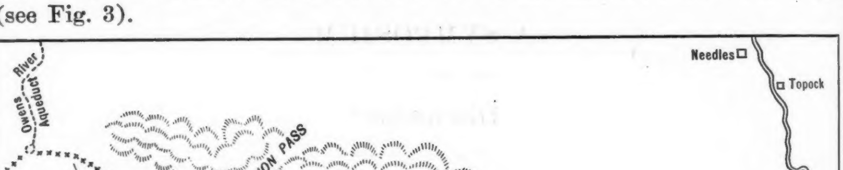

Fig. 3.—Map Showing Los Angeles-Colorado River Aqueduct.

Of these, the most southerly, and, therefore, the ones of particular interest to San Diego, Calif., would divert from the existing Imperial System in the vicinity of Dixieland and would rise to the crest at an elevation of nearly 4 000 ft. above sea level in less than 40 miles. Even with power at 0.5 cent per kw-hr., water so delivered would cost San Diego less than that secured by the present storage system. As this diversion would deliver into coastal

* Cons. Engr., Los Angeles, Calif.

† Received by the Secretary, April 23, 1928.

‡ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1116.

streams which flow into large existing reservoirs of that city, it could be put in without reference to Colorado River storage and the plant could be operated only when excess flow was available.

The next northerly possibility is a diversion from the Imperial System in the vicinity of Westmoreland into a conduit running west through the Borego Valley and crossing under the summit in a long tunnel under the Town of Julian. This route would vastly supplement the water supply in the vicinity of Oceanside, but would not reach the urban district around Los Angeles to advantage. It would also be deficient in storage.

The most northerly of the Imperial Valley routes would lie through the Imperial and Coachella Valleys to a regulating basin west of Indio at about Elevation +100. This point would be almost 130 miles from Laguna Dam across a smooth unbroken plain of soft soils well suited to the excavation of an earth canal. As the revised crest at Laguna will be at least as high as Elevation +170, there would be ample fall available.

In reaching this point, the All-American Canal and its extension northwesterly would serve more than 500 000 acres of land, and incidental to that service the water would be completely desilted before the first pumping plant was reached. From this terminal reservoir of the Imperial System the water would be lifted by boosters through a series of canals lying in the more elevated portions of the valley until it passed over the Beaumont crest of San Gorgonio Pass into the coastal areas. No canal would require tunnels or extensive rock excavation.

During the first half of the development period, or until at least a flow of 1 000 sec-ft. was in actual use, even the summit lift would be without reduction by tunnel. For these, sections could be located so that power recovery plants could be placed cheaply on the westerly slopes and return fully one-half of the gross power. This run in high-level canals and pumping lines would not exceed 50 miles in total length.

The Beaumont summit is at Elevation 2 500 and is 80 miles in an air line from Los Angeles. In the route suggested by Mr. Van Norman this 80 miles is covered by a continuous high line conduit passing directly through many of the smaller cities en route. However, that route possesses no present or future possibilities for the storage essential to the certain delivery of 1 000 000 acre-ft. of water per year 270 miles from its diversion from a natural channel.

Better economy can be had by spilling at the summit southwest into the San Jacinto River. Along the channel of this stream below Elevation 1 500 and continuing to Lake Elsinore, there is a series of possible storages with a great aggregate possible capacity, and of extraordinary safety since the basin is closed at Elsinore by a natural dike and not by an artificial dam.

For the Los Angeles Metropolitan Supply, at least, the water may again be spilled into the available natural channels and may be recovered only 36 miles from Los Angeles in the narrows of the Santa Ana River at Elevation 500.

This Imperial-San Jacinto route requires a total of only 216 miles of construction, no indispensable tunnels, and less than 50 miles will require

lining. The gross power consumption for full flow would be increased 25% over the Van Norman route, but with the same length of summit tunnel it will be only 5% greater. The time of construction will be fixed by the delivery of large pumps and the assembly of canal excavation equipment rather than by the time to drive tunnels. The initial cost will be predicated on the initial flow to be handled since the earth canals can be enlarged as the demand grows.

If, at some later date, a full gravity system is substituted, the loss of fixed investment will be limited to the abandoned pumping plants and such high-level canals as do not fit the proposed new elevation. The earth canals in the Imperial Valley would be released to purely irrigation use.

Tentative estimates indicate that the cost along this line would not be more than one-half or one-third that along the Blythe route; and here, again, as in the San Diego line, the possible coastal or delivery end storage would be so great that the line could be operated regardless of storage conditions on the Colorado River. Indeed it could be operated during some years of light load with off-peak power from the existing systems, or the 70 000 kw. which may be made available from the All-American Canal without reference to the completion of the power plant at the Boulder Canyon Dam.

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PAPERS AND DISCUSSIONS

PROGRESS REPORT OF SPECIAL COMMITTEE ON FLOOD-PROTECTION DATA

Discussion*

By Charles A. Pohl, M. Am. Soc. C. E.

Charles A. Pohl,† M. Am. Soc. C. E.—A discussion of flood protection should include an inquiry into the question of the transportation of soil by stream currents. This phenomenon manifests itself in erosion of banks, sweeping away of bars, formation of new channels, etc. The resulting deposition of this material in new bars, builds up the foreshore and banks, fills, reservoirs, and areas of low velocity with silt and forms deltas at the mouths of rivers.

These actions may be partly prevented by proper engineering constructions. Bank protection of various types helps to prevent the enormous caving of river banks. On the Mississippi, the amount of material caved has been known to reach more than 4 000 000 cu. yd. per mile of bank per year.‡ Most of this material is probably deposited as new bars a few miles down stream to build up the banks, while the finer and more colloidal materials may even be carried out to sea. The deposition may be controlled, to some extent, by side contractions, effected by means of permeable dikes and hurdle dams which break the current and allow settlement in the slack-water thus formed.

On the Mississippi River a minimum figure as to the amount of transported material is available from annual surveys of eroded banks. This, however, takes no account of material brought in from surface streams and larger tributaries.

Mr. G. K. Gilbert has shown§ that the action of débris in a stream of water is either one of pure rolling along the river bottom, a leaping from one point to the next along the river bed (called saltation), a pure suspension in the

* Discussion of the Progress Report of the Special Committee on Flood-Protection Data continued from May, 1928, *Proceedings*.

† Cons. Engr. (Bogart & Pohl), New York, N. Y.

‡ H. R. Doc., No. 50, p. 4.

§ "The Transportation of Débris by Running Water," by Grove Karl Gilbert, *Professional Paper No. 86*, U. S. Geological Survey, p. 15.

water stream, or a combination. An important distinction between saltation and suspension is to be noted. Mr. Gilbert calls them "hydraulic traction" and "hydraulic suspension", respectively. He has published the results of many experiments on hydraulic traction and the factors governing them. The formation of bars is largely caused by the coarser material.

Hydraulic suspension covers equally important and perhaps more complicated phenomena. Three general classes may be enumerated, each merging into the other:

1.—Fine material which is kept in suspension by the action of water currents and which will settle when the current is decreased sufficiently and for a sufficient length of time.

2.—Suspensoids, microscopic and larger particles, which are kept in suspension, perhaps by electric charges, and which settle readily by the action of an electrolyte, such as sea water, mineral-bearing ground-water, trade wastes, etc.

3.—Colloids of ultra microscopic size which may or may not be sensitive to an electrolyte.

Experiments conducted by the U. S. Corps of Engineers in 1879* showed that the floating sediment coming from the Missouri River into the Mississipi was 187 000 000 cu. yd. during nine months of the year. Taking into consideration the heavy material near the bottom, which was not included in the estimate, it was thought safe to assume that there was fully 400 000 000 cu. yd. emptied from the Missouri River in a twelve-month period. Other observations indicate a similar outflow from the Ohio, Arkansas, and Red Rivers, of 36 000 000, 5 000 000, and 6 000 000 cu. yd., respectively.

Experiments were made by Mr. G. F. Deacon, in Manchester, England, in 1883, on the transporting power of streams, from which he concluded that the amount of material transported varied as the fifth power of the current velocity.† At St. Louis, Mo., the mean velocity of the Mississippi River at the 35-ft. stage is about three times the mean velocity at the 5-ft. stage, and its transporting power would thus be 243 times as great. Floods have gone higher than the 41-foot stage at St. Louis with correspondingly greater velocities and carrying capacities.

This may or may not be a correct indication of the transporting capacity, but it indicates that at higher stages the material transported may depend more on the resistance to erosion of the banks and bed of the stream than on the capacity of the stream itself. It is further probable that rarely, if ever, is the capacity of a stream for transportation fully satisfied. It is reasonable to expect that, in seasons of great floods, the material transported by the Missouri River into the Mississippi may be at a rate much in excess of that estimated for 1879.

No less an authority than the late James B. Eads, M. Am. Soc. C. E., reported on conditions discovered during the flood of 1851 at a depth of 65 ft. in the Mississippi River at Cairo, Ill. Captain Eads, who was in a diving bell,

* Annual Repts., U. S. Chf. of Engrs., 1881, p. 1653, and 1887, p. 3000.

† *Minutes of Proceedings,* Inst. C. E., Vol. CXVIII, pp. 93-96.

recalled that the sand under his feet was driven by current almost as fast as at the surface. Even 2 ft. below the surface the motion was marked, but with a diminishing velocity.

Other instances of scour are reported* at depths of 100 ft. and 78 ft., in the latter instance when the river was only 10 ft. above its low-water elevation. Similarly, at the abutments of the St. Louis Bridge, the bed-rock was found to be smooth and water-worn, indicating scour to that depth at one time. Captain Eads found that a rise of water to 13 ft. below high water created a scour of 18 ft. He concluded that in the Mississippi the floods removed material almost, if not quite, to the bed-rock itself.

This confirms the contention† of J. F. Coleman, M. Am. Soc. C. E., that with increasing river stages the bed is scoured to greater and greater depths, giving not only increased discharge area from the rise in water surface, but also from the lowering of the bed.

The U. S. Government Special Board of Engineers, to which the speaker was a consultant, reported on two dams across the Mississippi River for power purposes, one at Jefferson Barracks, near St. Louis, and one above the Thebes Bridge.‡ The Board concluded that the transportation of silt was such a problem as to make any of these developments impractical and that the pools created by these dams would be silted up to such an extent as to raise the flood-plane to a height that would make the flood damage prohibitive.

T. U. Taylor, M. Am. Soc. C. E., shows§ that, in the seven years preceding the destruction of the first Austin Dam, the lake had silted up nearly 50% and in the thirteen years since the construction of the second dam, the silting has filled more than 95% of the volume of the lake.

Mr. E. F. McCarthy‖ calls attention to the almost complete erosion of soil in areas where vegetation has been killed by smelter fumes near Ducktown, Tenn. Evidence of this erosion is given by the considerable, if not alarming, extent to which reservoirs down stream have been silted. The Upper Tennessee River and its tributaries are generally stable as to bank erosion yet in numbers of instances silting in reservoirs and quiet areas outside the main channel has averaged more than 1 ft. per year. This condition also exists on the Atlantic slope of the Southern Appalachian Mountains and in some instances to an alarming extent.

The speaker has seen the Las Animas River at Trinidad, Colo., with a flow of about 100 cu. ft. per sec. at noon when a storm was beginning. At 3:00 P. M., the flow had increased to a raging torrent of perhaps 2 000 cu. ft. per sec., and at 6:00 P. M., the flow was substantially the same as at noon. The soil in the drainage area is largely hard-baked adobe, with steep slopes and with practically no vegetation. Much erosion took place on the ground surface, in arroyos and stream beds and banks. In the speaker's judgment,

* "The St. Louis Bridge", by S. M. Woodward, M. Am. Soc. C. E.

† "Levees as a Means of Flood Control for the Mississippi River," *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2550.

‡ H. R. Doc., No. 762.

§ "The Silting of the Lake at Austin, Texas," *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 569.

‖ "Forest Cover as a Factor in Flood Control," *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2511.

had the soil been reasonably protected by forests or other vegetation, the crest of the flood would have been materially delayed and lessened, and the erosion would have been reduced partly from the protection given by the soil covering and partly from decrease of the maximum volume of flow in the stream beds.

The deposition of colloid and suspensoid matter is effected by the addition to the stream of mineral salts, acids, trade wastes, etc. The discharge of acid wastes into the river from the smelters near Ducktown, may, and probably does, contribute to a more rapid and excessive deposit of fine suspended matter in the areas immediately below.

It is quite certain that silt deposition at the mouth of the Mississippi River is hastened and aided by the electrolyte in the sea water as it diffuses into the fresh silt-bearing water. Trade wastes from the manufacturing centers on the Ohio and its tributaries may, when they diffuse with the silt-laden waters of the Mississippi at Cairo, be the cause of additional deposits below the junction.

These comments are not intended to give a complete discussion of the silt problem, but to call attention to certain existing data and observations which bear on the subject. Complete and conclusive opinions cannot be given without much more data, collected in a uniform and systematic manner. It is hoped that the references given will serve to show the need for the gathering and publication of it in a collected form so as to be useful in flood-control problems.

The following points are suggested as bearing on this phase of the flood-control problem and as having particular reference to the Mississippi River:

1.—Collection of data as to amount of erosion and nature and size of material carried.

2.—Collection of more data with particular reference to the relation of discharge, velocity, and slope of stream to material carried.

3.—Study of stream flow as to the effect of electrolytes in solution on deposition of silt.

4.—Study of stream beds to determine relation of lowering of stream beds with increase of stage.

5.—Preparation of a standard form for collecting information so that complete and necessary data will be obtained in each instance.

6.—Beginning a systematic collection of data by competent people as to the effect of reforestation on stream flow, capacity of springs, and soil erosion.

7.—Attempt to interest hydraulic engineers in a more complete and thorough study of the whole problem.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION ON DRAINAGE OF IRRIGATED LANDS

Discussion*

By M. R. Lewis, M. Am. Soc. C. E.

M. R. Lewis,† M. Am. Soc. C. E. (by letter).‡—This report is of great interest and, as a brief summary of the subject, is certainly of permanent value. It shows careful thought and a broad understanding of the problem. However, there are a few instances in which the writer believes it should be modified or elaborated.

In the section devoted to the "Cause of Necessity for Drainage",§ the influence of alkali is omitted. The almost universal presence of excessive quantities of soluble salts in irrigated areas necessitates much deeper drainage than is common in humid areas. Unless the water-table is lowered to such a depth that only negligible quantities of water will be brought to the surface and there evaporated, a concentration of alkali salts will occur. Where the drainage is shallow and, as a result, the water-table is only 2 to 4 ft. below the surface, the rise of alkali can be controlled only by complete flooding of the surface by irrigation water.

In most sections this water carries in solution comparatively large quantities of soluble salts. Frequently, the quantity of most of these salts exceeds that carried off by the harvested crops. As a result, in the absence of all downward percolation to the water-table, there will be a gradual concentration of alkali salts in the surface soil. Even where the water-table is at a satisfactory depth below the surface, this fact makes it essential that some water be allowed to percolate downward through the soil to the ground-water. This water also must be removed by under-drainage, either natural or artificial.

* This discussion (of the Report of the Committee of the Irrigation Division on Drainage of Irrigated Lands, presented at the meeting of the Irrigation Division, Denver, Colo., July 14, 1927, and published in March, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Corvallis, Ore.

‡ Received by the Secretary, April 9, 1928.

§ *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 132.

For this reason, even if it were possible, it would not be advisable to irrigate with exactly the quantity of water which is used by crops.

In the discussion of the water-holding capacity of irrigated soils, two factors have become confused. Tables 3* and 4† purport to give the capacity of various types of soils to retain moisture. Actually, they give some combination of the capacity to retain moisture and the ability to absorb it under field conditions. The water-retaining power of soils is progressively greater as their texture becomes finer. (See Fig. 1.‡)

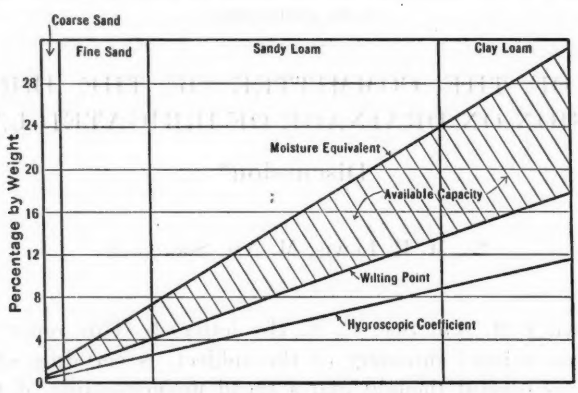

FIG. 1.—MOISTURE-HOLDING CAPACITY OF SOIL.

There is considerable evidence that the moisture equivalent is a very fair measure of the field capacity of a soil, both relatively and absolutely. While it is difficult to demonstrate this fact in heavy clay soils on account of their impermeability, the conclusion seems reasonable that fair agreement will be found, at least in surface soils which are not under considerable pressure.

It is true that under field conditions it is difficult if not impossible to make the heavier clay soils absorb as much as they can retain. However, it is best to keep clearly in mind that this is a problem of application of water and not one of its retention. As given in the report, Table 4 would indicate almost as great danger of over-irrigation of clays as of sands. In fact, of course, there is no such danger.

In the reclamation of black alkali lands both the Oregon and the Idaho Agricultural Experiment Stations have secured promising results by planting sweet clover on the undisturbed surface of the ground and irrigating very frequently. As is stated in the report, further studies of this problem are needed, and they are being made.

Control of the use of water by the farmer is a more complicated problem than is indicated in the report.§ Not only must the capacity of the soil to hold moisture be known, but also its ability to absorb that water. In tests

* *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 136.

† *Loc. cit.*, p. 137.

‡ Adapted from "Outline of Ground-Water Hydrology," by O. E. Meinzer, U. S. Geological Survey, *Water Supply Paper 494*.

§ *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 142.

on alkali lands the writer found that at the end of 1 hour the soil at one spot absorbed water at the rate of 0.16 ft. per hour, whereas at a point 75 ft. away, in the same border strip, the rate was 0.019 ft. per hour. Other tests on a supposedly uniform loess soil at Moscow, Idaho, showed a rate of infiltration three times as fast at one point as at another 5 ft. away. The problem of the rate of infiltration and application to the field of the facts discovered is a pressing one.

The use of pumping for drainage appeals to the writer as being the greatest single forward step made by irrigation engineers in many years. He expects to see its application very greatly extended, primarily because of the apparent ability to lower the water-table by this method to much greater depths than is feasible by either open or closed drains of the gravity type.

In this connection it may yet be found that "drainage solves the alkali problem" at least to a very great extent; but it must be real drainage, not just a feeble lowering of the water-table below the surface.

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PAPERS AND DISCUSSIONS

REPORT OF THE DUTY OF WATER COMMITTEE OF THE IRRIGATION DIVISION ON THE CONSUMPTIVE USE OF WATER IN IRRIGATION*

Discussion†

By Harold Conkling, M. Am. Soc. C. E.

Harold Conkling,‡ M. Am. Soc. C. E. (by letter).§—This report on the consumptive use of water in irrigation will be welcomed by the great body of engineers in Western States who are engaged in the problem of water supply for irrigation and domestic use. About 1918 the writer was engaged in a series of studies undertaken by the U. S. Reclamation Bureau for a comprehensive and complete development of some of the major stream systems of the West. The Rio Grande, North Platte, Humboldt, and Colorado Rivers were the principal streams studied.

It was soon apparent that as far as the water supply was concerned, the first effort must be to determine the consumptive use for different areas. The quantity that it was necessary to divert was a detail for any particular project which governed the size of the works, but was not usually a factor in determining the possibilities of a stream system, since consumptive use is not greatly influenced ordinarily by the volume diverted. It also seemed probable that more water is consumed per acre in the warmer climates than in the cooler and some of the streams studied, notably the Rio Grande and the Colorado, furnish water for lands in climates ranging from the high cold valleys of Colorado to the sub-tropical heat at the Mexican border.

A thorough search for data on consumptive use was made because, before the investigations could proceed intelligently, this basic matter of water sup-

* The membership of the Committee on Duty of Water of the Irrigation Division is as follows: S. T. Harding, *Chairman,* Harry Barnes, Lynn Crandall, Augustus Griffin, Charles R. Hedke, O. W. Israelsen, R. I. Meeker, H. M. Murdock, R. L. Parshall, W. L. Powers, G. E. P. Smith, and O. L. Waller.

† This discussion (of the Report of the Duty of Water Committee of the Irrigation Division on the Consumptive Use of Water in Irrigation, presented at the meeting of the Irrigation Division, Denver, Colo., July 14, 1927, and published in April, 1928, *Proceedings*) is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

‡ Chf., Div. of Water Rights, State Dept. of Public Works, Sacramento, Calif.

§ Received by the Secretary, June 6, 1928.

ply had to be settled. Very few determinations of consumptive use had been made at that time; some of those used by the writer are included in its report, modified by later information. It was believed at the time that those determinations, which were made by measuring the total quantity of water reaching a considerable tract of irrigated land and the total quantity leaving the tract, were the most reliable as they gave directly the average result of all the multitudinous factors that may affect the consumptive use on any individual acre. Accordingly, an investigation of all the Reclamation Bureau projects was made to find such areas as were favorable to determination in this way, but such tracts were very scarce. The writer does not now know whether or not the Reclamation Bureau has followed this up.

Since 1923 the writer has had charge of an investigation of the water supply in San Gabriel Basin of Southern California. The term, San Gabriel Basin, applies to the entire San Gabriel stream system. The streams of this system, like most of those in Southern California, flow across detrital filled valleys of unknown depth, and these valleys form huge reservoirs from which the water supplies of the region are drawn. Without these reservoirs the life of the region as it is to-day would be impossible because long series of dry years are characteristic of the climate. A determination of the consumptive use in the upper of the two detrital filled valleys across which the San Gabriel flows was made by balancing inflow against outflow. In that region—because of the large quantity of water pumped from the underground reservoir—change in underground storage is a much larger and important item than in the usual determination of this nature in the inter-mountain country. Any attempt to convert the measured change in elevation of water-plane into acre-feet of storage may result in serious error. In this investigation a determination of voids in various classes of materials encountered by the well driller was made and then from well-logs the percentage of voids at each well was calculated. With many well-logs to use, a map showing lines of equal voids was made. When there was superimposed on this map the data showing lines of equal annual change of ground-water level, a calculation of the acre-feet change could be made. Even with this elaborate study, chance for large error exists and unless the water-plane has moved in both directions there is no check to determine such error. In this case the average water-plane fell two years, remained stationary one year, and rose one year. The final results were such that it is believed the determination of acre-feet put into or withdrawn from underground storage each year is reasonably accurate.

Another factor which affects the apparent consumptive use in any one year is the large difference in rainfall. This affects it in two ways. If the rainfall is deficient the growth of vegetation on unirrigated land is less than normal and, consequently, the actual consumptive use is less. Furthermore, it is probable that, in such a year more water is released to the water-plane from that part of the vertical section of the valley fill above the water-plane and below the root zone than reaches it from deep percolation of rainfall. Hence, there is a depletion in this area which cannot show in the hydraulic equation and the apparent consumptive use is less than the actual. This

possibly is a considerable item as the ground surface over a large part of San Gabriel Valley is 200 ft. or more from the water-plane. It is believed, however, that after two years of nearly the same precipitation, conditions will have reached an approximate state of equilibrium. Data for the period since 1923 are not yet complete, but in Table 9 are shown data as to inflow and outflow for four years, the first two of which were very dry and the second two slightly wetter than normal. It is believed therefore that the second and fourth year average should give results that eliminate, to a large extent, the uncertainties noted in the foregoing. By a coincidence the average for four years is about the same as for the two years in question.

One other unknown quantity enters into the determination. The underflow out of the valley is uncertain. The approximate cross-section of the pervious area of the valley outlet is known and the approximate underflow is computed from this knowledge. While the variation from the figure used may be large, it does not seriously vitiate the figure for total consumption. The hydraulic equation is arranged in Table 9 to show the different items entering into it. The resulting "apparent consumptive use" is found by subtracting the total output from the total supply to the valley floor.

It will be noted that the apparent consumption is 191 000 acre-ft. This is believed to be close to the actual consumption, but for various reasons is probably a little larger than the actual (value assumed for underflow treated as definitely known). This supplied the water consumption of 129 600 acres of valley land, of which 81 700 acres are using water (irrigation or domestic), 33 200 acres are irrigable, and 14 700 acres are non-irrigable (country roads and washes). In addition, water is pumped to 3 240 acres of hillside land. Crops are produced as follows: 30 334 acres of citrus trees; 19 758 acres of deciduous trees, mostly English walnuts; 8 831 acres of truck gardens; 2 960 acres of alfalfa; and, 23 111 acres of urban area. Adding the land irrigated on the hillsides to that irrigated in the valley gives 84 940 acres irrigated or using domestic water.

If the 191 000 acre-ft. consumed is divided by the gross area, the consumptive use is 1.44 acre-ft. per acre. Does this value compare with that given in Column (9*a*) of Table 7* of the report? Nothing is said there as to whether the irrigated areas given in Table 7 are net or gross, but from the writer's knowledge of these various tracts he is inclined to believe that the acreage given is the gross and that the net irrigated is from 75 to 85% of the gross.

To obtain consumptive use in San Gabriel Valley on a comparable acreage basis, it is necessary to estimate what the increased consumption will be when the entire irrigable land is placed under cultivation. The fallow land consumes only rainfall, while the irrigated land consumes both rain and irrigation water. To determine this increased consumption it is necessary to find the differences in the ability of native and cultivated crops to capture rainfall. These differences are more profound than is generally supposed, and work is in progress at present, which tends to show that in the areas of heavier precipitation the difference in total water consumed by native vegetation and

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, pp. 238–239.

by cultivated crops is not great, except in the case of alfalfa and the area of this crop in the valley is negligible. In the absence of final determination on this point it is here assumed (for the purpose of finding a figure approximately comparable to that used in the report) that the consumption on irrigated areas is 1.9 ft. in depth and that on fallow irrigable land is 0.8 ft. in depth, so that when the valley is entirely irrigated the total consumption will be 36 000 acre-ft. greater than at present, making 227 000 acre-ft. in all.

TABLE 9.—CONSUMPTION OF WATER IN SAN GABRIEL VALLEY, IN ACRE-FEET.*

	SEASON.				
	1923-24.	1924-25.	1925-26.	1926-27.	Average.
INFLOW:					
Measured Inflow:					
Mountain run-off	32 470	30 060	141 000	163 000	91 600
Precipitation on valley floor	114 000	128 000	218 000	251 000	178 000
Total measured	146 470	158 060	359 000	414 000	269 600
Unmeasured Inflow:					
Mountain run-off	2 200	2 100	9 700	11 200	6 300
Hill run-off on surface	1 860	1 960	7 260	8 590	4 920
Lateral percolation from hills	0	0	5 750	6 610	3 090
Total unmeasured	4 060	4 060	22 710	26 400	14 310
Total inflow	150 530	162 120	381 710	440 400	283 700
OUTFLOW:					
Measured Outflow:					
Storm water through Narrows	2 380	4 530	52 800	81 900	35 300
Storm water through Arroyo Seco	0	0	240	1 000	310
Pumpage through Narrows	14 800	14 500	12 900	12 200	13 600
Rising water through Narrows	73 200	56 000	49 000	57 200	58 850
Sewer discharge through Narrows	70	4 220	5 130	5 320	3 680
Total measured	90 370	79 250	119 870	157 620	111 790
Unmeasured outflow:					
Storm water through Monterey Park	200	200	2 000	2 000	1 100
Underflow through Narrows	25 000	25 000	25 000	25 000	25 000
Total unmeasured	25 200	25 200	27 000	27 000	26 100
Total outflow	115 600	104 450	146 900	184 600	137 900
Total inflow	151 000	162 000	382 000	440 000	284 000
Change in underground storage	−113 000	−129 000	+2 400	+58 700	−45 000
Total supply to valley above Narrows	264 000	291 000	380 000	382 000	329 000
Total outflow	116 000	104 000	147 000	185 000	138 000
Apparent consumption	148 000	187 000	233 000	197 000	191 000

*** The annual rainfall in the valley averages 19.56 in., of which 78% falls in the four months from December to March, inclusive.**

It is believed that this figure, divided by the gross valley area plus the land irrigated on the hillside will give a value comparable, so far as percentage of area irrigated is concerned, with those used in the report. When this is done it results in a consumption of 1.7 acre-ft. per acre.

While this value is believed to be comparable on the basis of acreage, it may not be comparable in other ways. It is a measure of the total water consumed, whether from irrigation or rainfall, during the entire year, whereas that given in the report is for consumption from rainfall and irrigation during the crop-year. However, if the rainfall had not occurred in San Gabriel Valley, it is believed from other data that consumption of irrigation water (if irrigation had continued through the twelve months) would not have been greatly different from 1.7 acre-ft. per acre. In explanation, it should be stated that the growing season extends throughout the entire year and that cessation of irrigation occurs only because of the winter rains.

Referring to Fig. 2,* wherein "heat units" are compared to "valley consumption", it is found that this value for San Gabriel Valley would plat a little lower than that for the Kings River and the Kaweah River areas on approximately the same location as to "heat units". Examination of the data from which this graph was platted shows a substantial disagreement between values from the Truckee and Snake River areas and those for similar climates, such as the Boise, Cache la Poudre, Sevier, and South Platte Valleys. The reason for the excess consumption in the Truckee Valley may rest in the combination of the peculiar underground conditions and the excessive applications, but this does not apply to the Snake River area. An examination of the indirect methods used in securing the data for this area shows that many errors could occur.

The Committee raises the question as to whether a curve of heat units against consumptive use is justified.† It may be that it is, if it is restricted to showing the relation between the consumptive use of areas growing essentially the same crops. The crops of the inter-mountain country are mainly alfalfa, grain, and roots, and are essentially the same from north to south, but nothing quite comparable is found in California except in the extreme northern part or in the small areas east of the Sierras. In most places the irrigated crops are different and contain a larger proportion of trees. Then, too, humidity is less in the inter-mountain country than it is west of the Sierras. It would seem improbable that a comparison of consumptive use based on heat units is logical between such divergent types of crops. If the Kings and Kaweah values (see Fig. 2) are neglected and only the values from the inter-mountain region considered, the curve becomes almost vertical. If the Snake River and Truckee values are given little weight—the first because of possible inaccuracies and the second because of unique conditions—there is little basis for a curve. If one is drawn it could as well be in a much different direction than the one which appears as Fig. 2.

The addition of the San Gabriel Valley to the group, its rather close check with the other California areas, and the fair consistency in the type

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 241.

† *Loc. cit.*, p. 231.

of crops grown in these areas, suggest that while heat units may be a determinant in increasing the consumptive use, the higher type of crop grown in the hotter regions where land values are high requires less water. Therefore, if marketing conditions are such as to justify the higher type of crop, and high land value, a curve of "consumptive use-heat relation" will not be of value when applied to all the different conditions encompassed in the data published by the Committee.

The work of the Committee is, and will be, of great value. To the writer the report is made difficult reading by the use of symbols and equations in the nomenclature which do not seem to be justified. The subject is not mathematical in any but the arithmetical sense. However, this does not detract from the value of the Committee's work and the fair and careful manner in which the material is presented.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PROGRESS REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION ON FILTERING MATERIALS FOR WATER AND SEWAGE WORKS

Discussion*

By Messrs. J. W. Armstrong, Harry N. Jenks, and J. N. Chester.

J. W. Armstrong,† M. Am. Soc. C. E.—The report of the Committee seems to cover the things the speaker wanted to say, to a very large extent, but those who are familiar with the operation of filters know that at times filter runs are materially shortened, and the shortening of a filter run cannot be accounted for altogether by the turbidity of the water. This and other reasons led the Committee to believe that perhaps the influence of the character of water should be given greater consideration in the selection of a sand size.

Consider, for instance, the City of Providence, R. I., which has a soft water, highly colored. It is probable that a sand size suitable for Providence would not be suitable at all for waters received from the Great Lakes or from cities having large impounding reservoirs where the water has a high organic content. Again, it is believed that a different size of sand would be required in the waters of the Middle West which, at times, run very clear and at other times very turbid.

Fourteen cities located in different parts of the country have agreed to co-operate with the Committee in conducting a series of experiments to determine, if possible, the optimum size and depth of filter sand. Cities were selected with the purpose of securing water of various kinds, that would be typical for different sections of the country. Each city was furnished a plan

* This discussion (of the Progress Report of the Committee of the Sanitary Engineering Division on Filtering Materials for Water and Sewage Works, presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 19, 1928, and published in April, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Filtration Engr., Baltimore City Water Dept., Baltimore, Md.

and is to build and operate a battery of seven small glass filters that will be run in conjunction with one of their large filter units.

In order to eliminate every possible unknown factor and to insure the operation of each plant under identical conditions, the sand will be graded and sent to the operator. Each of the small units will be filled with a different size or depth of sand, and the water to be filtered will be withdrawn from the influent pipe of one of the large filter units.

In order to find out whether the glass filters behave in all respects the same as the large unit, one of them will be used for control purposes. Sand will be taken from the large unit and placed in the control unit, of the same depth and grading as that in the large unit. Should the results of the control and the large unit differ in any respect, a correction factor can be determined for use in interpreting the result of the experiments. The response of the cities co-operating was hearty and they are bearing their own expense. There will be very little cost to the Society for these experiments.

Harry N. Jenks,* Assoc. M. Am. Soc. C. E.—The investigations of this Committee are not only of unusual value, but are also very timely. From his experience in charge of the operation of the filtration plant at Sacramento, Calif., and because of his present participation in sewage treatment research at Iowa State College, the speaker is one of a large group of sanitary engineers who recognize the importance of the information which the Committee is seeking to obtain for the use of the profession.

Through force of circumstances at the Sacramento plant, it became necessary to operate one battery of filters with 30 in., and another battery with 24 in., of sand. Over a prolonged period of time there was no appreciable difference in quality of the effluent in respect to either bacterial count or brilliancy. A study was made to ascertain any possible difference in behavior as regards air-binding, from which it was concluded that the lesser depth of sand was not subject to as much air-binding as the greater depth of bed. However, with the approximately 0.4 mm. effective size of sand used in the filter beds, the influence of depth was not at all marked within the range mentioned. It will be of great benefit to designers if the Committee can determine minimum permissible depths corresponding to given sizes of sand, which would yield satisfactory effluents at normal rates of filtration. In general, it would be in the interest of economical design to know how deep to make a bed of sand with material of a given size and uniformity; or, conversely, what size a sand should be to yield the desired results for a given depth.

In continuing his studies of filter behavior under various operating conditions, the speaker had occasion to make a detailed study of the question of filter runs in relation to the rate of filtration. It was found that, for the plant under consideration, the product of the filter runs times the rate of filtration above normal was equal to some constant. It occurred to the speaker at the time, that such a relationship might be of general application and might also lead to a clue as to the effect of quality of water on filter runs,

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with a given size and depth of sand. A study such as the Committee proposes for determining the influence of varying quality of the applied water, as measured by its physical characteristics, should be of significance in defining allowable rates of filtration consistent with satisfactory length of filter runs. The physical features of the filter influent are doubtless the most important in this regard, because it appears proper to consider that the sanitary quality of the filter effluent is dependent primarily on the adequacy of pre-treatment rather than on refinements in the specifications relating to the filter sand.

Referring now to the investigations the Committee is making on filtering materials for sewage works, it will be of interest to note that the Engineering Experiment Station at Iowa State College has begun an extensive series of studies along these lines. For the present, attention is being confined to industrial wastes treatment and the effect of different sizes and classes of material used in trickling filters for packing house and sugar beet wastes. This field of investigation is a very broad one, involving as it does considerations of such factors as concentration of the sewage, its chemical and physical characteristics, rates of dosing, depth of bed, and temperature conditions. From the designer's point of view, the results of such an investigation would approach the ideal if the inter-relationships of these factors could be expressed in a definite, quantitative form, whereby the permissible rate of application of the sewage on a trickling filter, for example, could be closely ascertained from values assigned to the independent variables. Thus, as judged by the criterion of quality of effluent, these inter-relationships might be capable of expression in the following form:

$$Q = f\left(\frac{T\ A\ H}{C\ S\ E}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

in which,

Q = permissible amount of sewage to be applied to the filter in a given time.

T = temperature of liquid.

A = area of bed.

H = depth of bed.

C = concentration of organic matter applied to the filter.

S = size of filtering material.

E = quality of effluent as measured by percentage reduction of original organic concentration.

Equation (1) sets forth solely a functional relationship existing among the factors indicated. The problem is to determine empirical coefficients and exponents that would permit what might at least approach a direct solution of the resulting equation defining the dependent variable, Q. Such an ideal expression may be impossible of attainment; nevertheless, it is offered here by way of suggesting a fruitful line of investigation leading to a better understanding of the principles of trickling filter design. Supplementary studies needed to make such an expression of practical value would involve the defining of limiting sizes of filter material, S, which would give reasonable assurance of freedom from clogging by the wastes in question.

Mention of the subject of clogging brings to mind a promising field of research looking toward the possible use of relatively fine filling material for trickling filters. In connection with a current project of the Engineering Experiment Station of Iowa State College, the speaker is collaborating in a study of means whereby a filter may be readily "de-clogged". A practicable method of this kind would permit the use of such filter materials as might prove to be biologically the most efficient, regardless of their tendency to become clogged.

Considering, further, another aspect of the problem of the selection of filtering materials, the matter of durability is well recognized as of prime importance; but information on just how durable a stone must be to withstand a given sewage is as yet far from being as complete as it should be. It is well understood that certain wastes, notably those from industrial establishments, may be quite deleterious to a grade of stone that would otherwise be quite suitable for use in an ordinary municipal sewage treatment plant. A typical cause of disintegration is due to the acid constituents of the waste, as measured by the pH of the liquid. On the other hand, in a well oxidized effluent from either an industrial waste or a domestic sewage treatment plant, the pH is normally well above the point where trouble from an acid reaction is encountered. Such a gradual increase in pH-value (decrease in acidity) is typical of the biologic transformations taking place within a filter bed. In passing from below the neutral point, where the value of the pH is 7.0, to a point above, where the reaction becomes alkaline, the danger from active acid attack of the stone is passed. This suggests the possibility of using an acid-resisting material in the upper portion of the filter, overlying a somewhat less durable, and presumably considerably cheaper, material comprising the lower layers of the bed. Experiments involving a study of this method of design are in progress at Mason City, Iowa, where the field work of the research project already mentioned is being done.

The speaker believes that the results of research in the field of filtering materials for water and sewage works will fully justify all the painstaking efforts that may be made to correlate and extend the present knowledge of the subject. It would be gratifying to the speaker and his associates if they could be of some assistance to the Committee in investigating the problem in hand, with particular emphasis on the relationship of size and character of filtering material to biologic efficiency and to the engineering suitability of the various filtering materials available to designers.

J. N. Chester,* M. Am. Soc. C. E.—It seems to the speaker that no amount of experimenting or standardizing can account for, or compensate for, changes that are likely to take place in many of the water and sewerage plants of the country. The problems of the plants along the Great Lakes, afford a simple illustration, especially at the approach of what is generally called the "annual turnover", which occurs when the waters become sufficiently warm at the surface so that there is a disturbance that causes extreme air-binding. If the adjustment of sand, whether it is the size of the grains, the uniformity of

* (The J. N. Chester Engrs.), Pittsburgh, Pa.

grains, or the depth of bed, can mitigate the troubles with which the filter men along the Great Lakes are confronted at that time, they would be very grateful and very much surprised.

Take as another example a plant such as the one at East St. Louis, Ill. Most of the year this plant treats Mississippi River water, but an occasional heavy output of the Missouri River a few miles above, with a low Mississippi, causes a large percentage of Missouri River water to enter. The result is a change from the coarser sediment in the Mississippi River, which is readily coagulated, to the finer, silty make-up of sediment from the Missouri River that is difficult to coagulate.

Those instances—the first on the Great Lakes where air binding is set up by that "annual turnover" and the second in East St. Louis where a radical change takes place in the quality of the original turbidity—grade off in other ways. It has been stated that, beginning at Providence, R. I., where the water is clear, soft, and frequently highly colored, the turbidity becomes coarser and coarser toward the West and, in localities, changes from clay to loam. The Missouri River water is in a class by itself. These considerations add problems that are not going to be materially corrected by the size or the uniformity of the grains or the depth of the bed; that is, the two abrupt differences, such as "the turnover" in the Great Lakes and the changing waters of the Mississippi and the Missouri.

In a study of the size of sand and its depth for use in water-works filters of the rapid sand type, it would appear that some common ground must be found upon which to base such studies. What is the paramount factor guiding such studies? Shall it be purely economics? That is, (1) shall it be based on considerations as to the effective size and thickness of sand bed; the quantity of wash water used; the length of runs; the quantity of chemicals required and their character; and the maintenance charges coincident with removing mud balls and periodically cleaning the sand; or (2) shall it be the quality of water to be derived from the filter? It may be a consideration of both these factors. As an example, one of the foremost State health departments requires a filter product which will pass the U. S. Treasury Department Standard without depending on the chlorine treatment for assistance. In other States the filter is not required to meet such a standard, and greater dependence is placed on the chlorine treatment for effective reduction of the bacteria. The size of the sand used in the filter of the first of these conditions necessarily must be finer than that required for the second condition. The chemical treatment and the character of chemicals used are frequently determined by the personal equation, and although such factors have an important bearing on the successful filtration of any waters, the result of proper treatment will not depend entirely on the size of the sand grains or the depth of the filter bed, but will be determined largely by the kind of supervision given all these factors by the operators.

At Wheeling, W. Va., it has been developed that a change in the kind of chemicals used, which resulted in a material saving in the chemical bill, also resulted in materially increased filter runs with a consequent saving of wash

water. This was accomplished by diligent study and followed by experimentation by the chemist. In view of the tremendous variation in the character of the waters to be treated in the United States and the varying requirements and ideas of the health departments and engineers and operators, if the Committee can only generalize in its conclusions as to the proper size of sand and depth of the bed to be used under various conditions, it will have accomplished a great deal and will deserve the gratitude of the members of the Sanitary Engineering Division.

MEMOIRS OF DECEASED MEMBERS

JOSIAH FOSTER FLAGG, M. Am. Soc. C. E.*

Died April 13, 1928.

Josiah Foster Flagg was born in Dedham, Mass., on September 4, 1835, the son of Josiah Foster and Mary (Wait) Flagg.

From the Boston Latin School he entered Harvard University, at Cambridge, Mass., from which he was graduated in 1854 with the degree of Bachelor of Science, *Summa cum laude.* Immediately following his graduation he was employed as Rodman and Draftsman on the Cleveland, Columbus and Cincinnati Railroad.

In the spring of 1855, Mr. Flagg returned to Boston, Mass., where he was employed as Assistant Observer and Computer at the Harvard Observatory, having prepared himself for such work before graduating from College. While engaged in astronomical observations he went on one of the expeditions to England, in charge of fifty-two chronometers, for the purpose of checking the difference in longitude between Greenwich and Washington. This engagement led to his appointment as Assistant to Lieutenant Gisliss, at the Washington Observatory, in reducing the astronomical observations which the latter had previously made in Chili.

In the spring of 1857 Mr. Flagg resigned from the Observatory Staff to become Assistant Engineer under General (then Captain) M. C. Meigs, in charge of the United States Capitol and General Post Office extensions, and also of the Washington Aqueduct. He continued in this position until the close of 1859 when deficient appropriations necessitated a general reduction of the staff.

In the spring of 1860 he became Assistant Engineer to the late James B. Francis, Past-President and Hon. M. Am. Soc. C. E., in charge of the locks and canals at Lowell, Mass. This work, however, terminated toward the close of the year.

When the Civil War commenced Mr. Flagg offered his services to the nation in a civil engineering capacity. Not being successful in his patriotic efforts in Massachusetts, he went to Washington to further his claim for service. While he was there, the Battle of Bull Run occurred and in the tense situation that followed, further effort seemed hopeless.

In August, 1861, he accepted a position as Draftsman with Bement and Dougherty in Philadelphia, Pa. At that time this firm was second in importance to that of William Sellers and Company as manufacturers of machine tools. Later in the same year he was sent by this firm to the Springfield Armory to study the methods of making muskets, machine tools for quantity production of small arms being in great demand at that time.

In March, 1862, Mr. Flagg resigned this position to join the Atlantic and Great Western Railroad Company, subsequently the New York, Pennsylvania and Ohio Railroad, at Meadville, Pa. For ten years he served this

* Memoir prepared by James R. Chapman, M. Am. Soc. C. E.

Company in an engineering capacity as well as Land and Tax Agent and sometimes as Paymaster.

In March, 1872, he resigned this position to take charge of the erection of machinery for an extensive nitrate plant at Iquiqui, Peru. This work required two years for completion. In 1874, Mr. Flagg assisted in the examination of the Magdalena River in Venezuela in connection with a project to improve it for navigation.

Following his return to the United States in 1875, he was one of three mechanical experts engaged to test steam engines and other machines entered in competition at the Cincinnati Exposition. The tests and compilation of the reports took several months.

In January, 1876, Mr. Flagg returned to Meadville, Pa., as City Engineer, taking up the private practice in his profession at the same time. He occupied this position for five years, resigning in 1881 to take charge of the construction of the Pacific end of the Mexican National Railway, with headquarters at Colima, Mexico. During the following three years he located, constructed, and operated 130 miles of line mostly in a very rough and difficult country. The fact that the office of the General Manager was in New York and that of the Chief Engineer in the City of Mexico—a ten-day journey—left Mr. Flagg in a position of great responsibility. All construction material and rolling stock were received by vessel. Right-of-way details and the disbursement of all funds were in his charge. Due to financial difficulties which caused a suspension of construction, he resigned in 1884.

After some months spent in travel, Mr. Flagg went to Cuba in the fall of 1885, to report on proposed plans for water supplies for the Cities of Santiago and Guantanamo. After returning from Cuba he was placed in charge of the up-town works of the New York Steam Company, in New York, N. Y.

Subsequently, Mr. Flagg was employed by the Engineering Commission appointed to formulate plans for sewage disposal at Washington, D. C. He was in charge of the investigations and computations necessary for the prosecution of the undertaking.

At the conclusion of this service he traveled extensively in Europe and North Africa making many investigations relative to sewage disposal as developed in England and on the Continent. The latter years of his life were spent in retirement in Santa Barbara, Calif., where he died.

Mr. Flagg was married on April 13, 1858, to Emma A. Wiggin who died in May, 1888. One daughter was born to them—Helen B. Flagg who died in 1902. On December 3, 1903, he was married to Florence N. Dukes, of Brooklyn, N. Y., who survives him.

Mr. Flagg was elected a Member of the American Society of Civil Engineers on October 7, 1874.

AUSTIN BRADSTREET FLETCHER, M. Am. Soc. C. E.*

Died March 8, 1928

Austin Bradstreet Fletcher was born at Cambridge, Mass., on January 19, 1872. His father, Ruel H. Fletcher was, during a long and active life, a

* Memoir prepared by A. N. Johnson, M. Am. Soc. C. E.

teacher in the public schools of Cambridge. His mother was Rebecca C. (Wyman) Fletcher. They were descendants of English stock, whose ancestors came to Massachusetts in early Colonial times.

Mr. Fletcher prepared for Harvard University at the Cambridge High School, and entered the Lawrence Scientific School in the fall of 1889. After two years of work in the general Science Course, he took courses in Civil Engineering, and was graduated in 1893, receiving the degree of Bachelor of Science.

Immediately after his graduation from college, he took a position as Secretary with the newly organized State Highway Commission of Massachusetts, and remained in the employ of this Commission until 1910. He had not been long in active work before he gave evidence of that exceptional executive and administrative ability for which he became noted. At the time he left the Massachusetts Highway Commission he was performing the duties of Executive Officer and Chief Engineer of the Commission.

The State Highway work in Massachusetts was pioneer work in this field, and attracted nation-wide attention, both for the actual work accomplished, as well as for the organization developed. Great credit was given Mr. Fletcher for the smooth-running, efficient organization that he perfected, the personnel of which was maintained for more than twenty years. This alone presents a great contrast to the many changes which have so often characterized public works enterprises.

During the time he served the Massachusetts Highway Commission, this organization was given supervision over the long-distance telephone lines along the public highways, which in consequence greatly broadened the scope of his duties.

At this period the character of highway traffic was undergoing a great change. It was evident that the past methods of construction would be insufficient to withstand motor traffic. The great problem presented was how most economically to treat the roads already built in which large sums had been invested, and also how best to build the new roads. No one foresaw these changes and the new demands better, or felt the responsibility more keenly, than did Mr. Fletcher. The many experiments and methods of treating road surfaces that were carried on by the Massachusetts Highway Commission attracted the widest attention. At the time Mr. Fletcher severed his connection with the Commission he was established as the foremost engineer in the highway field.

In 1910 he was induced to accept the position of Chief Engineer of the Highway Commission of San Diego County, California, where a construction program involving millions of dollars was planned. He was engaged in this work scarcely a year when he was appointed the first State Highway Engineer of California. Here, he was destined to accomplish the greatest and most comprehensive program that has fallen to the lot of any highway engineer. One of Mr. Fletcher's colleagues in this work pays the following well-deserved tribute:

"So well did he install system in California that few changes have been made, or perhaps can be made, in the engineering principles and standards

devised by this farseeing pioneer of highway development. His ideas and ideals will ever remain foundation stones of the organization structure of the State Highway Department of California".

While engaged as State Highway Engineer of California, Mr. Fletcher was called in consultation, on several occasions, by the United States Bureau of Public Roads. During the summer of 1916, he assisted in drafting the rules and regulations to carry into effect the Federal Aid Road Law, which had just been passed by Congress.

Following the death, in 1918, of Logan Waller Page, M. Am. Soc. C. E., Chief of the United States Bureau of Public Roads, Secretary Houston, of the Department of Agriculture, tendered this position to Mr. Fletcher, who declined it, preferring to carry on the great road program in California, in which he was engaged.

From 1917 to 1923, in addition to his highway duties, he was President of the California State Reclamation Board, and from 1921 to 1923, he was Director of Public Works of California. In January, 1923, he was called East to act as Consulting Engineer for the New England Rail Committee. This Committee was made up of representatives of six New England States to study the whole transportation system in New England. Mr. Fletcher made a study and submitted a report on the motor-truck transportation phase of this investigation.

From September 1, 1923, until his death, Mr. Fletcher was connected with the United States Bureau of Public Roads as Consulting Highway Engineer.

In 1908, he was sent, as delegate from Massachusetts, to the First International Road Congress, held in Paris, France. He attended the Third International Road Congress held in London, England, in 1913, as delegate from California. In 1914 he was chosen President of the American Road Congress, which was held at Atlanta, Ga., in November of that year. During the World War he was a member of the Executive Committee of the California Council of Defense.

He was the author of a *Bulletin* on Macadam Roads issued by the United States Department of Agriculture in 1906. He contributed the chapter on "Drainage" in the American Highway Engineer's Handbook, and published many papers in various technical journals.

Mr. Fletcher was a member of the American Society for Testing Materials, the American Association of Engineers, the American Concrete Institute, the Boston Society of Civil Engineers, the Massachusetts Highway Association, the American Road Builders' Association, the American Association of State Highway Officials, the Permanent International Association of Road Congresses, and a Fellow of the American Geographic Society.

He took a great interest in and was a member of a number of historical and patriotic societies, among which were the New England Historic-Genealogical Society, the Massachusetts Society of Sons of the American Revolution, and the Society of the Colonial Wars of California. He was a member of the Cosmos Club, of Washington, D. C., of the Harvard Club of San Francisco, Calif., and of the Harvard Engineers Club, of New York, N. Y.

Mr. Fletcher died on March 8, 1928, at his home in Chevy Chase, Md. He had just returned from a business trip to Cleveland, Ohio, where he contracted a severe cold that developed into pneumonia.

On March 1, 1894, he was married to Ethel Hovey, of Cambridge. There were two children, Dorothy, now Mrs. Laurence H. Chapman of Sacramento, Calif., and Norman, who died when but seven years of age.

Although Mr. Fletcher was shy, sensitive, and modest to a fault, he was, nevertheless, an exacting and firm administrator. He wasted little time on what he considered useless argument and speculation; he was not to be stampeded. He had that essential quality of all great executives, the ability to select a capable personnel, and to inspire and maintain in his organization that spirit of loyalty and enthusiasm aptly called *esprit de corps*. While he was severely just in his dealings with subordinates, his disciplinary measures were always tempered with a kindliness that left no heart-burnings.

Those, of the organization which he perfected in California, speak of him with the greatest respect and loyalty. In the April, 1928, number of the Official Journal of the Department of Public Works of California, dedicated to him, there is paid this testimonial to his all too short, but useful, life:

"Mr. Fletcher lived to see the tree of his life mature and fruit in an added happiness and an enlarged usefulness given to the whole people of California. What greater monument could any one build? What greater reward could any one ask?"

Mr. Fletcher was elected a Member of the American Society of Civil Engineers on June 1, 1909.

GEORGE WASHINGTON GOETHALS, M. Am. Soc. C. E.*

Died January 21, 1928.

George Washington Goethals was born on June 29, 1858, in Brooklyn, N. Y. the son of John Louis Goethals. His family was of Dutch extraction. He entered the College of the City of New York at the age of fourteen, and remained there for three years. He was then selected for a cadetship at the United States Military Academy by the Hon. S. S. Cox, Representative in Congress of the 6th District of New York. At West Point, Mr. Goethals was much liked by his classmates and other cadets, and maintained a prominent standing in his studies and his military duties. He was a Cadet Captain in his first-class year, and was elected President of his Class. Upon graduation in 1880, he was one of two to receive commissions in the Corps of Engineers of the United States Army.

After serving for two years at the Engineer School and with the Battalion of Engineers at Willets Point, N. Y. he was appointed Engineer Officer of the Department of the Columbia, with station at Vancouver Barracks, Vancouver, Wash. He then served for a year as Assistant to the late Col. W. E. Merrill, Corps of Engrs., U. S. Army, M. Am. Soc. C. E., on the works of

* Memoir prepared by H. F. Hodges, M. Am. Soc. C. E.

improvement of the Ohio and tributary rivers. From 1885 to 1889 he was Instructor and Assistant Professor of Civil and Military Engineering at the United States Military Academy, and from 1889 to 1891, Assistant to the late Col. J. W. Barlow, Corps of Engrs., U. S. Army, on the work of improvement of the Tennessee and Cumberland Rivers.

During the next three years Captain Goethals was in charge of the Tennessee River improvement and the completion of the Muscle Shoals Canal, including the design and construction of the Colbert Shoals Lock. There, he improved his previous insight into problems of canal construction and of administration and direction of work executed by the Government without intermediary contractor.

From 1894 to 1898 he was Assistant in the office of the Chief of Engineers, U. S. Army, in Washington, D. C. At the outbreak of the Spanish-American War he was appointed Lieutenant Colonel of Engineers in the Volunteer Service and assigned as Chief Engineer of the First Army Corps. As such, he organized and constructed the water supply system of the camp at Chickamauga, Ga., and directed the engineering operations of the Corps in the Porto Rican Expedition.

At the conclusion of the war he reverted to his regular rank of Captain and was appointed Instructor of Practical Military Engineering at West Point, being also in charge of completing the water supply system of the Post and the reconstruction of the Library Building. He was promoted to a Majority in February, 1900, and served in charge of works of river and harbor improvement and fortification in Rhode Island and Southeastern Massachusetts, until the creation of the General Staff of the Army under President Roosevelt and Secretary of War Root, in 1903. Major Goethal's outstanding qualifications prompted his selection on the first list of this body of picked men. His tour of duty with the General Staff had almost reached the statutory limit of four years when, as a Lieutenant Colonel, he was selected by President Roosevelt to be Chairman and Chief Engineer of the Isthmian Canal Commission, succeeding John F. Stevens, Past-President and Hon. M. Am. Soc. C. E., who had tendered his resignation. At the same time radical changes were made in the remaining personnel of the Commission and all the members were required to live on the Isthmus of Panama in immediate touch with the field work.

The year ended December 1, 1906, was stated by the then existing Commission in its last Annual Report to have virtually closed the stage of preparation. In that stage, plant for excavating and other work had been purchased or ordered on liberal lines, commensurate with the extent of the project. A railroad transportation system had been organized which was ample and well suited to dispose of the spoil from Culebra Cut. A good plan of recruitment of labor had been put into operation and an adequate supply assured. In matters of sanitation, quarters, and subsistence, a great deal had been accomplished, and wholesome living conditions had been created for the large and increasing forces. Good progress had been made with the preliminary work

upon which the final designs for the canal structures, as then planned, could be based.

By the time the new Commission, with Lt.-Col. Goethals at its head, took charge, on April 1, 1907, the excavation was well started and a considerable volume had been removed from the canal prism in Culebra Cut and from the site of the Gatun Locks. Nevertheless, serious and difficult problems of plan, organization, and execution, vital to the success of the work, remained to be solved. The designs for the locks, dams, and accessories had to be adopted definitely and pushed to rapid completion; the manner of carrying out the work, whether by contract or hired labor, a question which was still open, had to be decided; the plant to mix and lay millions of yards of concrete had to be designed and built; certain matters which were considered as settled, such as the width of the Culebra Cut, the position of the locks and dams near the Pacific Terminal, the width of the locks, the location of the new Panama Railroad, and the position of the Atlantic breakwaters, had to be re-opened and the original decisions changed. The great terminal piers, the Pacific dry dock, the coaling plants, and the permanent shops were so far in the future that they had not even been included in the plan; the actual work of excavation and construction had to be carried on and completed; the constructing organization changed from time to time to suit the advancing work; and, finally, an organization had to be developed for operating the canal and its subsidiary activities after completion.

Whereas the chief credit for the construction of the Panama Canal rightly belongs to the man who was for seven years its Chief Engineer and under whom the greater part of the work was done, the Canal itself will always be a monument to the genius and ability of the American engineer. Many members of the Society and many others whose names do not appear on its rolls, contributed to the successful result and have a share in the honor.

By Executive Order of January 6, 1908, President Roosevelt restated the duties of the Commission, placing an increased measure of authority and responsibility in the hands of the Chairman individually; and from that time forward no one doubted who was in control of the work, and no one in the Canal force doubted its ultimate success.

The achievements of the next seven years are already well-known to those who will read this memoir, and require no extended recounting here. The Canal was finished within the estimate of cost made in 1908, and was opened to traffic before the date then set. Operation, however, was interrupted later for some months by slides in the Culebra Cut, but the Canal has since been serving a heavy traffic which is now threatening to tax its capacity.

Toward the conclusion of the construction period, when the organization for operation was developed, Colonel Goethals was made the first Governor of The Panama Canal. On March 4, 1915, he was made a Major General of the Line of the Army and received the thanks of Congress for his services. On November 1, 1916, he was retired from active service at his own request.

In 1917 he was appointed State Engineer of New Jersey by Governor Edge. After a few months spent in laying out a new highway system for

New Jersey and in practice as Consulting Engineer, he was appointed by President Wilson to be General Manager of the Emergency Fleet Corporation, United States Shipping Board. He was pronounced in his opposition to the plans for extensive construction of wooden ships, and resigned from the Fleet Corporation in July, 1917, to resume his consulting practice.

In December, 1917, General Goethals was recalled to active service and made Acting Quartermaster General, United States Army. In this capacity and as Assistant Chief of Staff and Director of Purchase, Storage, and Traffic, he served through the remainder of the World War, and until March 4, 1919, when he was relieved at his own request from further active duty. He then again took up his practice as Consulting Engineer in New York City, a practice in which he continued until his death. During its course he was associated for a time with the firm of Jamieson, Houston, Graham, and Jay, subsequently absorbed by George W. Goethals, Inc., of which Company he was President. He was also President of Goethals, Wilford, and Boyd, Inc., and of the American Ship and Commerce Corporation.

General Goethals and his associates were consultants on many important works, such as the Inner Harbor Navigation Canal at New Orleans, La.; the Columbia Basin Irrigation Project; the East Bay Municipal Utility District, of Oakland, Calif. and the Lake Worth Inlet District, in Florida. The New York-New Jersey Port and Harbor Development Commission, created by Acts of the State Legislatures in 1917, selected him as its Chief Consulting Engineer, and he continued as such with The Port of New York Authority created by the same Legislatures in 1921 to carry into effect the comprehensive plan prepared by the earlier Commission for re-organizing the facilities for, and modernizing the methods of, handling the commerce of the Port of New York. His connection with this great work continued until his death. In January, 1923, he was appointed Fuel Administrator of the State of New York, and served as such for a few months.

General Goethals possessed in a marked degree the technical qualifications, the good professional judgment, and the administrative and organizing ability which characterize the great engineer; but perhaps the most effective weapon in his armament was his tremendous driving power. This was not shown in arbitrary pressure or nagging criticism. Indeed, it would not be far wrong to state that it acted as much by inspiring as by spurring his subordinates; for he spared himself no more than he did them.

He succeeded in winning the confidence, respect, and even affection of the working force on the Isthmus, to a remarkable extent. Every one among them felt sure of fair treatment at his hands. His spirit rose especially against anything which savored of oppression of the lower by the higher; this characteristic was so well-known that all felt free to lay any case of real or fancied injustice before him in person, without fear of unpleasant after effects. His Sunday mornings were spent in such personal interviews. While they thus absorbed much of his scanty leisure time, and even more of his nervous energy and, perhaps, his patience, they were more than worth while, because of the intimate relation which they built up with the men in the ranks, and the consequent abiding trust in him which was the certain result.

General Goethals was a warm and steadfast friend to those to whom he gave his confidence; but this was not given readily, nor until he had proved to his satisfaction that it could be based upon esteem as well as personal regard. Once given, it was rarely withdrawn. Those who possessed it would have gone to any lengths for him.

He was a man of strong opinions, but open to argument and tolerant of difference from his own views, even of opposition to them, as long as a plan might be still under consideration. Once the matter had been decided and directions given, he expected loyal compliance, and did not welcome any attempt to revive the issue. To take such a step one had to have a very good reason and be sure of his ground.

For his service during the World War General Goethals received the Distinguished Service Medal from the President of the United States; the Cross of Commander of the Legion of Honor from the President of the French Republic; and was made Honorary Knight Commander of the Order of St. Michael and St. George by the King of England. He received also the Grand Cordon of the Order of the Striped Tiger, 2d Class, from the Chinese Government. He was the holder of numerous medals, among them, the John Fritz Medal "for achievement as builder of the Panama Canal"; the medal of the National Geographic Society; the Cullum Medal of the American Geographic Society of New York; the President's Medal of the Architectural League; and the medal of the National Academy of Sciences. He received Honorary Degrees of Doctor of Laws from Harvard, Yale, and Princeton Universities, the University of Pennsylvania, Johns Hopkins University, and Dartmouth College; of Doctor of Philosophy from the Chicago Polytechnic Institute; of Doctor of Science from Columbia and Rutgers Universities; and of Bachelor of Science from the College of the City of New York, which he attended before going to West Point. He was a member, regular or honorary, of many clubs and societies, social and technical, including the Institution of Civil Engineers of Great Britain and the Royal Engineering Society of Holland.

General Goethals' death was the occasion of many tributes to the high regard in which he was held. The President of the United States wrote that,

"General Goethals will be remembered most widely for his great achievement in the construction of the Panama Canal, as Chief Engineer, and in placing its operation and the administration of the Canal Zone on the extraordinary basis of efficiency which has made it so successful. But, his skill and genius were no less marked in other work. Particularly, were they invaluable to the Government during the World War when, coming back from well-earned retirement, he displayed his great ability in a variety of activities. His name was known throughout the world and will remain for all time on the roster of those who have done big things for our country."

The Governor of the State of New York wrote that in his passing,

"The State of New York, as well as the nation, lost a notable public servant. He brought to his duties as Chief Consulting Engineer of The Port of New York Authority that same expert wisdom which made him the genius of the Panama Canal.

"At great personal sacrifice he responded to my request to become Fuel Administrator of the State at a time when we were suffering a serious fuel emergency * * *. Here again, with his great executive talent, his power of decision and of direction, he gave to the State a useful and successful service.

"He has left behind a permanent memory of great personality and a splendid record of achievement."

Many other expressions from those who had been associated with him show how deeply they were impressed by his exceptional ability and by the strength of his character, the purity of his ideals, and the integrity of his motives. In the words of one of them, "He was at all times found equal to the important and responsible positions in which he was placed."

General Goethals was married in 1884 to Effie Rodman, of New Bedford, Mass. His widow and two sons, George R. Goethals, M. Am. Soc. C. E., and Dr. Thomas R. Goethals, of Brookline, Mass., survive him.

General Goethals was elected a Member of the American Society of Civil Engineers on March 1, 1910, and served as Director in 1918.

CHARLES WELLFORD LEAVITT, M. Am. Soc. C. E.*

Died April 22, 1928.

Charles Wellford Leavitt, the son of C. W. Leavitt and Sarah Allibone Leavitt, was born at Riverton, N. J., on March 13, 1871. He was educated at the Gunnery School, at Washington, Conn., and at the Cheltenham Military Academy of Pennsylvania.

Mr. Leavitt's engineering career began in 1888, when he accepted a position as Assistant Superintendent of Construction in his father's contracting firm. A year or so later, he became Assistant Engineer of the Caldwell, N. J., Railroad and was subsequently promoted to the position of Chief Engineer.

In 1892, he became Chief Engineer of the New York Suburban Land Company, engaged in the development of Essex Fells, N. J. Mr. Leavitt remained with this Company for three years, finally leaving it to enter the employ of the East Jersey Water Company.

In the fall of 1896, Mr. Leavitt formed a partnership with Rudolph Ulrich, a Landscape Architect, well known for his abilities. This partnership continued for about two years, at the end of which time Mr. Leavitt opened an office and established himself as a Civil and Landscape Engineer, concentrating on this work, which gave him so much pleasure and held so much of interest during his lifetime.

A review of some of the outstanding results of Mr. Leavitt's life work shows how diversified were his attainments within the scope of his field.

Among the larger private estates which were developed under his guidance were those of William C. Whitney, Daniel Lamont, Foxhall Keene, and Charles M. Schwab.

* Memoir prepared by M. W. Weir, M. Am. Soc. C. E.

Belmont Park, Saratoga, Empire City, Havre de Grace, and the Montreal Jockey Club are race-tracks on which Mr. Leavitt's planning had a controlling influence.

He made plans for the Palisades Interstate Park; Stamford Park at Stamford, Conn.; Monument Valley Park, at Colorado Springs, Colo.; Schenectady Park, Schenectady, N. Y.; Mahlon Stacy Park and the State House Park, at Trenton, N. J.; Saratoga Springs Park, Saratoga, N. Y., and for the Fairmount Park Commission of Philadelphia, Pa. He was also Landscape Engineer for the Camden County, New Jersey, Park Commission.

Among the Country Clubs which were developed under Mr. Leavitt's landscape plans were the Westchester-Biltmore, Rye, N. Y.; the Rumson Country Club, Rumson, N. J.; the Englewood Golf Club, Englewood, N. J.; the Lido Country Club, Long Beach, Long Island, and the New Orleans Country Club, New Orleans, La.

Many large real estate developments were planned and supervised by Mr. Leavitt, among which may be mentioned those for the Pennsylvania Steel Company at Steelton, Pa.; for the Worth Brothers, at Coatesville, Pa.; the Kilgour and Stettinius Estates, at Cincinnati, Ohio; Milner Heights, Birmingham, Ala.; and Long Beach, Long Island, N. Y.

University and institutional grounds also attracted his interest, and among those which he laid out were: Tome Institute; the New York Juvenile Asylum; the Peabody College for Teachers; Oglethorpe University; the Presbyterian College of South Carolina; Lehigh University; and Fordham University.

Among the cemeteries which Mr. Leavitt planned are the Gate of Heaven Cemetery, at Kensico, N. Y., and part of Woodlawn Cemetery, at Woodlawn, N. Y.

His city planning practice also included work done for Camden, N. J.; Garden City, N. Y.; Trenton, N. J.; Brunswick, Ga.; West Palm Beach, Fla.; Lakeland, Fla.; and Mount Vernon, N. Y.

Mr. Leavitt was a member of the American Society of Landscape Architects, Past-President of the American Institute of Consulting Engineers, a member of the Engineering Committee for the Plan of New York and Its Environs, the Architectural League, the National Conference on City Planning, the American City Planning Institute, the Interstate Metropolitan Planning Conference, the International Garden Cities and Town Planning Federation, the National Conference on State Parks, and the Garden Club of America.

He was a member of the Union Club, the Transportation Club, the Westchester-Biltmore Country Club, the Racquet Club, Philadelphia, the Adirondack League Club, the Play and Players Club, Philadelphia, and the New England Society of New York.

He was also a member of the New York State Chamber of Commerce, the Westchester Chamber of Commerce, the American Civic Association, and the New York State Board of Real Estate Boards.

Mr. Leavitt was a man of great energy and his interest in public affairs was keen, although he took no part in politics. Of clear mental vision, his judgment was sage and well founded. His imagination could always be

depended upon to look forward, and from this quality, coupled with business acumen and a driving personality, came, in a large measure, the successes for his clients in his chosen profession.

With these attributes, his congeniality, and his ready wit, Mr. Leavitt was a delightful companion, a splendid friend, and an inspiring leader. The influence of such a life cannot be estimated, but of this one may be sure, it was great, and its reflection will persist for the good of all.

Mr. Leavitt was married to Clara Gordon White on September 26, 1899, and he is survived by his widow and his four children, Gordon, Kent, Charlotte, and Dundas.

He was elected an Associate Member of the American Society of Civil Engineers on January 4, 1899, and a Member on May 2, 1905.

JAMES C. NAGLE, M. Am. Soc. C. E.*

Died April 6, 1928.

James C. Nagle was born in Richmond, Va., on October 9, 1865, the son of John and Ellen Mary (Smith) Nagle. His parents moved to Travis County, Texas, where his childhood was spent on a farm, and his primary education was obtained in a country public school near Austin. He received the degree of Bachelor of Science in Civil Engineering from the University of Texas in 1889, and the degree of Master of Arts in 1892. The same year he obtained the degree of Civil Engineer from the University of Pittsburgh and the degree of Master of Civil Engineering in 1893 from Cornell University.

Mr. Nagle's first professional experience was as Field and Office Assistant for the St. Louis Southwestern Railway Company in 1887. He was Assistant Engineer for the Austin and Northwestern Railway Company in 1888. In 1889 and 1890 he was employed as Assistant Geologist, chiefly on topography for a geological survey of Texas.

In 1890, he became Professor of Civil Engineering at the Texas Agricultural and Mechanical College and, in 1911, was appointed Dean of the School of Engineering.

In 1913 Mr. Nagle resigned this position to accept an appointment on the Texas State Board of Water Engineers. He served as the first Chairman of this Board from September, 1913, to September, 1917, when he resigned to return to the Texas Agricultural and Mechanical College as Professor of Civil Engineering, Dean of the Engineering School, Director of the Texas Engineering Experiment Station, and Consulting Engineer for the Prairie View State Normal and Industrial College.

Mr. Nagle served the Texas Agricultural and Mechanical College in these positions until September, 1922, when he resigned to devote his entire time to the business of the Nagle, Witt, Rollins Engineering Company, which he had organized in 1919 and of which he became the Senior member. He was still an active member of this firm at the time of his death.

* Memoir prepared by A. P. Rollins, M. Am. Soc. C. E.

From 1923 to November, 1927, Mr. Nagle was associated with R. A. Thompson, M. Am. Soc. C. E., on the investigation, design, and construction of a \$5 000 000 water supply reservoir for the City of Dallas, Tex.

In 1903, Mr. Nagle was married to Emily St. Pierre Davis (*née* Morgan), at Union, S. C. He is survived by his widow, two step-daughters, Mrs. Robert J. Potts, of Waco, Tex., and Mrs. F. F. La Roche, of Atlanta, Ga., a step-son, Major John F. Davis, U. S. A., of Fort Leavenworth, Kans., and two brothers, Judge M. Nagle, of El Paso, and John Nagle, of Manor, Tex.

In addition to numerous professional and technical papers, Mr. Nagle was the author of a "Field Manual for Railroad Engineers"; of three published reports on the "Silt-Carrying Capacity of Texas Streams"; and of *Bulletin 222,* on "Irrigation in Texas", of the United States Department of Agriculture.

He was a member of the American Association of Engineers, the Cornell Society of Civil Engineers, the Society for the Promotion of Engineering Education, the Southwestern Geological Society, and a Fellow of the Royal Society of Arts, London, England. He was also a member of the Protestant Episcopal Church, a Scottish Rite Mason, and a member of Ben Hur Temple, A. A. O. N. M. S.

Mr. Nagle devoted his life to his work in and for the State of Texas, and he was loved by all who knew him. The following tribute is quoted from the editorial column of the *Houston Chronicle* under date of April 8, 1928:

"Word that J. C. Nagle, former Dean of Engineering at A. & M., is dead, will bring regret to many thousands of men now active in the industrial life of Texas and the Southwest. This splendid engineer gave his talents for many years to training youths at the great Texas Mechanical College.

"His personal influence probably meant as much as the technical knowledge he imparted. He was a leader of men as well as a teacher. A State can never know what it owes to such men; their work lives in the lives of other men. Those other men know, however, and many of them will grieve to-day over the loss of their friend and mentor."

Mr. Nagle was elected a Member of the American Society of Civil Engineers on June 6, 1905.

FRANK ALEXANDER GIESTING, Assoc. M. Am. Soc. C. E.*

DIED APRIL 25, 1928.

Frank Alexander Giesting was born in San Francisco, Calif. on April 4, 1881. He was the son of Joseph G. Giesting and Clara Enneking Giesting. His youth was spent in the city of his birth, where he attended public and private schools until the end of his seventeenth year.

At that time, at the outbreak of the Spanish-American War, he enlisted as a private in the First California Heavy Artillery. Although perhaps the youngest member of his Company, he, even at that early age, exhibited such qualities of leadership that he arose almost immediately to the non-commissioned rank of Sergeant. He was chosen as a member of a small detachment

* Memoir prepared by Alva F. Hughes, Min. Engr., New York, N. Y.

from his Company to accompany the first Army Expedition to the Philippine Islands where he served with distinction until mustered out of the Volunteer Service in August, 1899.

In 1900 Mr. Giesting received a civil appointment in the Postal Department, at which time he returned to the Philippine Islands where he remained about two years. Resigning from the Postal Service in 1902, he returned to San Francisco and matriculated in the College of Civil Engineering of the University of California, Berkeley, Calif., at the beginning of the succeeding semester. He was graduated in May, 1906, with the degree of Bachelor of Science, and almost immediately began his professional career with the Progreso Mining Company at Triunfo, Baja California, Mexico, where he remained until July, 1907. At this time he was engaged for a short period as Transitman and, later, as Resident Engineer on construction for the Southern Pacific Railroad Company, then being built in Mexico.

In November, 1907, Mr. Giesting entered the service of the Mexican Light and Power Company, in which he acted first as Draftsman, then as Engineer and Superintendent on construction. It was during this engagement that he had complete charge of the construction of the famous Tanengo Tunnel (1 300 m. long) and the auxiliary works of this incline tunnel from Salto Chico to the power house. Coincidently, he was in charge of the 6 700-m. tunnel on the Laxaxalpan Diversion.

In July, 1912, he became General Superintendent for the engineering firm of Jacobs and Davies, Incorporated, which had taken over all the engineering work of the Mexican Light and Power Company, and served in that capacity until October, 1913. During the next two years he was engaged in private practice as Consulting Engineer in San Francisco.

From February, 1915, until April, 1917, he served as General Superintendent for the Aluminum Company of America in dam construction work at Alcoa, Tenn.

At the entrance of America into the World War, Mr. Giesting immediately resigned his position with the Aluminum Company and entered the Plattsburg Military Training Camp. His former military experiences, together with his obvious ability to lead men, resulted in his almost immediate transfer to the Training Camp for Engineers at Camp Humphrey, Va., from which he was graduated in the first class with the rank of Major of Engineers. His first assignment was at Camp Upton, Long Island, in the formation and building of which he gave prominent assistance. When the 302d Engineers of the 77th Division was formed, Major Giesting was assigned to that regiment. This regiment went overseas in March, 1918, and soon after its arrival in France, due to the promotion of his Senior Officers, he was made Lieutenant Colonel and given command of his Regiment. Under his leadership the 302d Engineers made an enviable record in the Argonne and were in the front line on Armistice Day. He had been given the rank of Colonel of Engineers and in that capacity returned to the United States with his Regiment in April, 1919, being mustered out of the service shortly thereafter.

After several months of rest, Colonel Giesting returned to the practice of his profession as General Superintendent of the Vanadium Corporation of

America from 1920 to 1921, and, subsequently, with the Engineering Department of the National Carbon Company from 1922 to 1924. His next engagement was with the Delaware and Hudson Railroad Company with which he was connected from 1924 until 1926. From April, 1926, to the time of his death, he was in the service of the Port of New York Authority as Assistant to the Chief Executive Officer.

Colonel Giesting's outstanding qualities of leadership and organizing ability marked him as an exceptional member of the Engineering Fraternity. In going over his record it is interesting to recall that in his capacity as Resident Engineer for the Mexican Light and Power Company he had complete charge of the construction of what at that time was regarded as the longest incline tunnel in the world; and, further, as to his military record, that as far as is known he was the only Reserve Officer in command of a Combat Regiment of Engineers during the World War. In his character, he was quiet and unobtrusive, and always generous to a host of friends who mourn his passing. He was a great Engineer and a great Soldier.

Colonel Giesting was elected an Associate Member of the American Society of Civil Engineers on October 1, 1913.

WALTER HARLAN LECKLITER, Assoc. M. Am. Soc. C. E.*

Died January 29, 1928.

Walter Harlan Leckliter, the son of John H. and Cornelia (Hanks) Leckliter, was born at Carbon, near Corning, Iowa, on November 19, 1883. He was graduated from the Corning High School in 1904, and from Iowa State College, at Ames, Iowa, in 1909, with the degree of Bachelor of Civil Engineering. In the interval between his Freshman and Sophomore years in college he was engaged with a surveying party on construction work for the Missouri Pacific Railroad Company, in Southern Missouri. He stood well in his studies at college and in his Senior year was elected to the Tau Beta Pi Fraternity.

After his graduation, in July, 1909, Mr. Leckliter entered the employ of the Chicago, Burlington, and Quincy Railroad Company, on maintenance work in Wyoming. He began this engagement as Rodman and was subsequently advanced to the position of Head of Party.

Previous to this time, he had taken and passed a civil service examination, and in May, 1910, he received a Government appointment to the Philippine Islands, where for three years he built roads at Baguio, the Summer Capital.

His work in the Philippines also included a several months' engagement on preliminary surveys for irrigation, highways, and bridge construction. In connection with the latter, he designed bridges and culverts of reinforced concrete and harbor improvements which consisted of docks, sea walls, and other similar structures, for the City of Samboanga. He was also in charge of the office of the Bureau of Public Works in Baguio, which office regulated and controlled all buildings, grounds, roads, water-works, telephone, and sewer-

* Memoir compiled from information on file at the Headquarters of the Society.

age systems, etc., for the city. He also had charge of an extension of the waterworks system, of which the estimated value was $20 000.

In 1913 Mr. Leckliter returned to the United States, and was employed as the first County Engineer of Adams County, Iowa. In this capacity he was engaged in road and bridge construction. In 1914, he entered private practice and began contracting for paving, with headquarters in Des Moines, Iowa. Later, he was employed as Superintendent of Construction with Akin and Flutter, General Contractors, until 1920, when he again engaged in private practice as a contractor, principally in grading and paving in Southern Iowa and Missouri. He continued in this work until ill-health necessitated his retirement in 1927. He died on January 29, 1928, at his home in Des Moines.

In his school days he was quite an athlete; he was Captain of his High School football team, although he never "made" the 'Varsity eleven. One of his college papers called him "a leader on the Campus". Besides Tau Beta Pi his only social memberships included a college Fraternity, "Adelante", and the local militia company at Corning.

Mr. Leckliter's character is perhaps best indicated by what other people thought of him. As a supporter of a widowed mother, and of a younger sister, whom he sent through college, he was conscientious to the largest degree. Even during his last illness, he was not immune from the demands of his wide friendship. One of his former laboring men wrote that he was out of work and tramping through Louisiana, and closed with "I know that if you only knew how much I needed money, you would send me some". At that time, he was besieged with financial demands because of the many expenses incidental to a long illness; yet he told his wife to mail the man a check.

One of his business partners has said of him, "There was a square man. I have traveled over all this country with him and there was nothing crooked in him." To his many friends, to his family, to whom he meant so much, his loss is great. Such a man must indeed be counted a success in life.

He was married on March 2, 1918, to Helen Widner, of Corning, Iowa. Besides his widow, he is survived by the following members of his family: His mother, Mrs. C. J. Leckliter, two brothers, Mr. Ralph E. Leckliter, of Corning, and Mr. Oliver L. Leckliter, of Des Moines; and four sisters, Mrs. Alma Bilderback, of Council Bluffs, Iowa, Mrs. Katherine Farrar, of Richmond, Calif., Mrs. Charles McGhee, of Steins, N. Mex., and Mrs. Arthur W. Miller, of LaGrange, Ill.

Mr. Leckliter was elected a Junior of the American Society of Civil Engineers on October 1, 1912, and an Associate Member on August 31, 1915.